DESIGN OF A GROUP OF MILITARY TIMBER BRIDGES EMPHASIZING LOGISTICAL ECONOMY

MICHAEL MOSTELLER

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by

MICHAEL MOSTELLER

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Submitted to the faculty of tens claer folytechnic Institute, Troy, New York, as partial fulfillment of the requirements for the degree of Master of 'cience in livil Ingineering.

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DESIGN OF A TROUP OF MILITARY TIMES TO SOME

I. IRTRODUCTION

A. Subject - The subject of this thesis is the development of a design for the more common military timber bridge structures which might be utilized by the U.S. Marine Jorps with the specific intent of effecting standardization to the fullest practicable extent.

B. History - Military briding operations follow a general pattern dictated by doctrine born of practical necessity. Then in the course of combat a stream crossing is enseuntered, the structure initially employed to provide more or less unrestricted vehicular passage is usually a prefabricated bridge such as the fixed panel type Bailey Bridge or the floating type ponton bridges us d so extensively in World ar II. These structures are desined with a view toward rapid erection under adverse combat conditions and adaptebility to a wide range of site conditions. After the advance has progressed forward sufficiently a semi-permanent bridge is constructed and the prefabricated bridge dismantled for further use in direct support of the combat operations. Short spen semi-permanent bridges are also frequently used in the improvement of main supply routes to cross narrow gulches and ravines or minor drainage channels. These semi-permanent bridges are commonly sade of timber due to its sase of fabrication with the tools ordinarily available to the constructing troops.

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In the past timber bridges have often been desi ned by the person directly in charge of its construction according to the cite conditions being confronted and the materials available to him at the time. This meant that the time required to design the structure occurred after the job was encountered, often as not the desi n was by "rule of thumb" processes, the desi n was forced to fit the available materials and the construction procedures were devised on the "individual problem" basis. These undesirable consequences were realily recognized and as a result at inderdization in certain respects was instituted to varying decrees at levels rangin from the construction unit to the engineer officer responsible in a liven area of operations. However standardization in the main has always been limited by availability of materials as opposed to making specific timber materials in grade, size, len th. etc. available according to the requirements of a standard desi n.

as practicable the so i-permanent timber bridges which are most company employed by the U. S. Marine Borps in military operations according to the varying demand of traffic capacity, load capacity and site conditions; and to determine the extent to which standardization of construction details, structural design and component materials required is feasible. In so doing it may be possible to improve efficiency in construction by training erection crews in the fabrication of standard joints and details, to produce the most economical but satisfactory design by deliberate predesign according

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to accepted engineering design methods, to reduce the time required to complete a bridging job by eliminating the bulk of design after job assignment, and to improve the efficiency of procurement, stocking and supplying timber materials that will meet the job requirements.

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II. WOODE

A. Types of Bridges - Though many types of bridges are used for semi-permanent installations in the combat zone, the timber treatle bride is by far the most prevalent. This is due to the fact that such a structure requires the least amount of material. it is most easily and quickly constructed and its suitability to a particular sito is not limited by the total span length of the crossing. The treatle bridge is applicable to those sites that are either dry or the streams are comparatively a allow, alow-moving and have a reasonably firm bottom. Fortunately these require ents are met in many crossings. In those instances where the nature of the site precludes the use of a trestle type structure, some type of truss bridge may be suitable. Nowever if the required truss is anything more than a simple short span truss, it is usually the practice to put in a fixed panel type brid's such as the Beiley for semi-permanent service. Inasmuch as the primary interest here is standardization, the types of bridges to be considered will be limited to those which occur frequently enough to cause standardization to be profitable; i. e., the timber treatle construction and simple trues bridges of limited span practicable for timber construction.

Since the structural design of a timber trestle bride is not a function of its total span length, there is no limit tion in span for this type of construction to which standardization will not be applicable. However in the case of truss type bridges only those span lengths will be investigated that can be constructed from timbers required in the trestle structures it being felt that loner

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the six sector already want is be considerable and the smallest state of the smallest and t

seans will be of such infrequent occurance that considerations of standardization will not be worthwhile. The limiting span for truss bridges will therefore have to be determined as the investigation proceeds.

B. Load Japacities - The nature and magnitude of loads to be carried by military bridges can be predicted fairly well because they will be used almost exclusively by stand rd military vehicles whose maximum gross weights and configuration are known. During the reater part of orld ar II it was co on practice to build main supply routes to a capacity of 35 tons per lane. This particular capacity limitation was due to the fact that the heaviest commonly encountered load was the "General Sherman" type tank, nominally a 35-ton vehicle. Loutes de anding heavier loadcarrying capacit, were infrequent enough and occurred at such places as to permit special consideration of briding roble s without the pressure of extreme military urgency. However the evolution in tank design during the latter part of orld ar II and since has changed the situation somewhat. First the Sh rman was modified to improve its fire power and in so doing its fighting weight increased to approximately 37 tons. Then the "entral Fatton" tank of ap roximately 46 tons gross fightin, weight was introduced. In the li ht of experience in the lorean war this tank expeers to be supplenting the Sherman as the principle armored vehicle for eneral purpose combat use. Therefore it seems that a route conscity overned by the loads imposed by the heavier latten tank will in the future come to be the usual requirement rather than the

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and address to the other part of the second - The state of the same of the same of the same of profit for the control of the contro And the second of the second o A latter with the same of the with the Control of the property of the party of the latter of the latte and the same the same to the s which was not to the same of t to be a second or the second of the second or the second o where the party of the same of the particular section to examine the same of the Committee of the second less reliable to the second less thanks AND REAL PROPERTY AND PERSONS ASSESSED FOR PARTY AND PARTY AND PARTY. Control to the control of the contro Designation of the latest section of the lat at the party of th will still be the control of the party of the party of were not the same of the party of the latter The first region of the control of the party of the control of the the United States of the United States and t

special case. There are relatively few vehicles employ d in combet by the Marine Corps between the 37-ton and 46-ton weight class and therefore on those routes which the Fatton tanks will not be used, the 57-ton capacity is still a reasonable upper limit to provide for the transit of all other common military traffic including the lighter Shermans. Hence from the point of view of standardization, bridges of two load capacities will be dealt with; that which will carry up to and including the Sherman tank and that which will carry the latton tank.

O. Traffic Capacity - Military bridges providing a mosas of stresm crossing generally have a maximum of two lanes; one in either direction. On many occasions single-lane brid es are built as is the case when the highway is limited to one-way traffic for military ressons. In those isolated instances where more than two traffic lanes are required at a single eroseing point, separate bridges are built sufficiently distant from each other to preclude complete traffic stoppage by a single hostile attack. For these reasons, the proposed standard design will include only single-lane and double-lane bridges. From practical considerations it is probable that the trues design will be further limited to single-lane bridges only.

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III. DEGIG CRIT IA

- A. General Insofar as ractical and except in those instances where modifications are dec ed necess ry because of military
 considerations, the American Association of the fit hway (fittials
 tandard specifications for highway rides and tion longs
 Specification for tress-grade Lumber and Its this swill overn.
- Design Vehicles Jince the light wride is to be de-8. si and socifically to pass the Thomas take as well any variete of equal or less gross witht it is as ropriate to un that tent as the desin vehicle (Pi . 1). It has a rose wi ht of 74,000 rounds distributed on two trac's that are F4 inches center to center. ech track is 16 1/2 inches wide with a round contact lenth of 147 inches. This re-ults in a uniform round pres ure of 15.25 pounds per inch for a length of 147 inches. Buch a desi n vehicle will pose the most severe loading with rerard to bending and abear in stringers and floor be s as well as stresses in bents a d trasses. However the well distributed natur of the los tu to the tricks does not produce a critical condition for tres is in the deck. Therefore it is necessary to sleet companion and 1 d which of equivalent gross weight to be used for lesign in this in those. There is no particular wheeled vehicle of approximately 37 tons ross weight whose use is sufficiently wiles read to worrant solection as the limiting vehicle to be passed by the light bridge. However the hypothetical II 20-3 16 truck of the A.A. .1.0. affords a whoeled vehicle of approximately the required weight magnitude. And the use of this loading for the deck design does not seem

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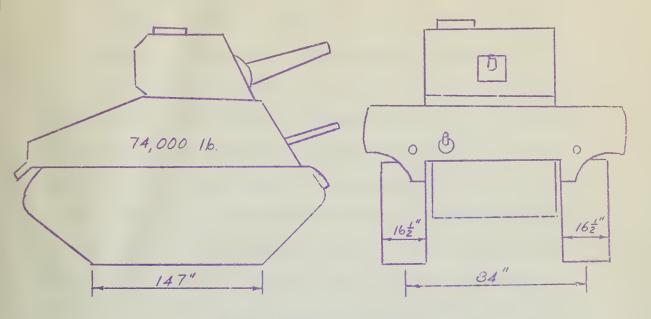


Fig. 1 Design Vehicle for Light Bridge

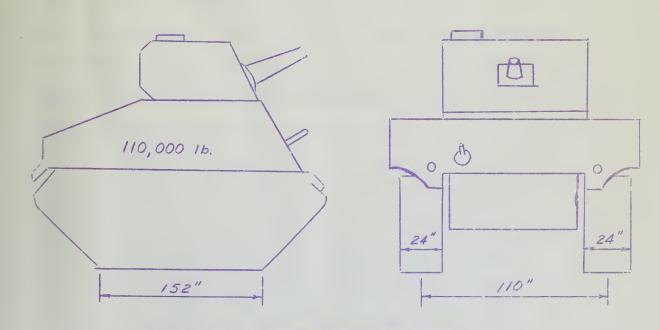


Fig. 2 Design Vehicle for Heavy Bridge



unreasonable insamuch as it produces a major wheel load of 16,000 pounds distributed over 20 inches of width as comp red to a wheel load of 14,900 rounds on an effective width of 26 inches found on one rarticular military vehicle in the 35-40 ten wei ht range.

Bimilarly the latton tank (Fig. 2) will be used as the design vehicle for the heavy bridge. However in order to anticipate future modification which inevitably result in wei ht increase, a gross wei ht of 110,000 pounds instead of the current fi hting weight of 92,500 pounds is considered more appropriate for design purposes. This tank has two tracks 110 inches center to center which are 24 inshes in width and have a ground contest length of 152 inches. It produces a uniform ground pressure of 15.08 pounds per square inch and a uniformly distributed load for each track of 352 pounds per inch. Again the companion wheeled vehicle for design will be a hyrothetical truck-tractor with semi-trailer of 108,000 pounds Such a design vehicle with a maximum wheel load of 24,000 pounds distributed over 30 inches of width compares favorably with 22,900 pounds on an effective width of 32 1/2 inches encountered on one particular military vehicle.

7. 'idth of Roadways - The required clear width between used timbers for the simple lane light bridge is detor ined by assuming that the eximum overall width of vehicle to use the bridge to be 102 inches and permitting a 24-inch arginal clearance at each side. This results in a clear width of 150 inches or 12 1/2 feet. For the double lane bridge two 102-inch vehicles are

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marginal clearance and a medial clearance of 36 inches between the.

Thus a total of 264 inches or 22 feet of clear width is required.

In the case of the heavy bridge the same clearances as used on the light bridge are applied but the design vehicle is taken as 158 inches in overall width. This requires for a single lane bridge 186 inches or 15 1/2 feet of clear readway and for a double lane bridge 336 inches or 28 feet of clear width is needed.

D. Other Design Leads - Dead load will consist of that portion of the weight of the structure by which any particular member is stressed. The unit weight of lumber will be taken as 40 pounds per cubic foot. This fi ure provides adequately for the use of any stress-rade lumber marketed in the United States which is in a dried state (15 to 18 per cent moisture content). Wominal dimensions will be used in computing dead weights as a matter of convenience since the error incurred is insignificant and dead load rarely affects the required size of member drastically.

Impact stresses will be computed as 30 per sent of the stresses due to live load. This follows the A.A.S.F.O. specifications which require that impact stresses be computed by the formula $I = \frac{50}{1. + 125}$

where I is stress due to impact, S is live load stress and L is the loaded span length in feet required to produce maximum stress. However the maximum impact fraction is limited to 30 per cent which would require that the loaded length be in excess of 41 2/3 feet to reduce the fraction. It is improbable that span 1 noths of such a magnitude will be used except in the longer truss bridges. Consequently

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the 30 per cent factor is both convenient as well as conservative.

No loads other than dead, live, wind and impact will be concidered.

E. llowable Unit Strasses - In order to ain full advantage of relatively precise engineering design, stress- rade lumber with a fixed allowable working stress must be utilized. Since allowable working stresses vary not only with species of lumber but also with the several grades of a given species, it seems advisable to develop the standardized design based on the species and rade most likely to be evailable in military o erations and them attempt to device s method for determining required member sizes when using lumber of a different allowable stress. Douglas Fir and Southern Pine are produced in greater volume than other domestic species and are therefore considered most likely to be available for procurement and ultimate use in combat areas. Furthermore it would not be fatal to use a higher grade lumber then required by the design whereas a lower grade would be dangerous. Consequently the selection of allowable stresses applicable to one of the lower grades of these two species would be a sound choice. Examination of the allowable unit stresses as specified in the National Design Specification for Stress-Irade Lumber and its Fastenings indicates that use of the following listed stresses

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the standard of the standard o not provide desperate action controlled makes y deplacing he alternative many provided to the market pulses with the later and the the man are second to enter the color of the first term and the color named of closeling some Properties with a Market Street and where you were the property and an investment of the party of the part married an expension with the application of the last particular and the the county species were come as the communication of the county of the c --- see one control or any section where the party of the party of neclearly the contract of the contract of the party of the contract of the con THE PARTY AND ADDRESS OF THE PARTY OF THE PARTY AND THE PA where the factor we are followed in proceedings, where the party of their the property of the party of th NAME OF PERSONS ASSESSED OF PERSONS ASSESSED. the first parameter and mostly by parties would not be pass and address. where the second pass of the officeral succession of the second second section in the U. Or widow Street Street Street Street St. of Street maked parenties our la car root probable? applicable? (7) has in the basic design will permit the safe use of most of the stress rades of Douglas Fir and Southern Pine:

Allewable Unit Stresses (pounds per square inch)

Extreme fiber in bending

Tension parallel to grain

Horizontal shear

Jompression perpendicular to grain

455

Compression parallel to grain 1150

According to the provisions of the National Design Specification
these allowable unit stresses are applicable for normal loading conditions. Normal loading is defined as the application of the full
maximum normal design load for a duration of approximately three
years or ninety per cent of the full maximum normal lesign load
continuously throughout the life of the structure without encreachin on the factor of safety. In these instances where the duration
of the load is limited, certain percentage increases are allowed
in the allowable unit stresses describing upon the length of time
the particular load is expected to be sustained.

As previously stated the proposed design will be based on the su port of dead, vehicular, wind end impact loads only. With regard to duration, dead load comes within the scope of normal loading conditions if the expected life of the bride is not over three years which is reasonable in military construction. Therefore the allowable unit stresses are applicable without any increase being permitted. Though the specifications permit an increase of 35 1/3 per cent for wind, Howard J. Hansen in his "Timber in incering handbook" indicates that for loadings not exceeding a duration of five minutes an increase of 50 per cent should be per issible and cites

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wind leads as coming within this category. This apparent inconsistency may be reconciled by the fact that the 33 1/3 per cent indicated in the specifications is actually the per itted increase for a lo d of eight hours duration whereas wind loads are ordinarily based on the hi hest sustained wind velocity for a priod of only five minutes as determined from data of the U. . . e ther Bure u. Consequently the specifications conservatively place wind loads in the eight-hour duration category while Hansen classes it mere properly as having a duration of five minutes and therefore worthy of greater increase. Accepting the plausability of a permissible increase of 50 per cent for loads of less than five minutes duration, such an increase can be justified for vehicular lo da since the stresses induced at a point in the structure may be considered as not persisting for periods in excess of five minutes if the vehicle maintains motion. The applicability of a 50 per cent increase for moving vehicular loads is further substantiated in publications of the Department of the Army dealing with design data for military timber bridges. In the case of maximum stresses due to impact, the specifications permit 100 per cent increase in the allowable unit strasses.

To recapitulate then, in the proposed design the allowable unit stresses previously selected will be subject to increases as indicated for eximum design loads of the following nature:

Pead Load - 0% Wind Load - 50% Vehicular Load - 50% I pact Load - 100%

when the same of t which have been did not seen that the party of the latter and the most record the contract of the contract o All received that were a control or and the second of the second of the second of the bearing a set provide the frequency could sell in head and the second s the same of the same of the same of the same of the street of the same of the party of the same of the the second or residence to receive a paint of thomas with the property of the property and property of the property A REAL PROPERTY AND ADDRESS OF THE PARTY AND A the same and account of latters, in our committee has the frequency of the electronic wife of policy of the families' presents ARTERS TO THE PROPERTY OF THE PARTY OF THE P and the second colors with the first production of the production of the second colors and the second colors and the second colors are the second colors are the second colors and the second colors are the second colors and the second colors are the second colors a to endposition of resolutions around all army reliables palme. the province of the roat will be the same out to convey all Married by the party of the par the property or of common payment All three probabilities and the same of th

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In conjunction with the use of steel gusset plates and boits in the joint details of the truss bridges, the following allowable stresses in steel will be used:

Allowable Unit Stresses

to the section making real by the o

(pounds per aquare inch)

Axial tension on net section Compression in splice material 24,000 shear for unfinished bolts with washers under nuts 13.500 Bearing, single or double shear, for unfinished bolts with wa hers under nute

28,125

These stresses have been taken from Department of the Arry publithe set of the set of cations an' thou h comewhat greater than thes found in American the law or committee out a property than the law of the Institute of Steel Construction specifications are in keeping with the practice of reducing the usual safety margin in military construction. The basic allowable stresses in shear and bearing for unfinished bolts as -iven in the military ref rences are 12,000 pounds and 25,000 pounds per square inch respectively. A further increase of one-eighth has been injected with the tipulation that washers will be used under all nuts in such a manner that the unthreaded shark of the bolt will extend fully through the gusset plates. This follows from provisions found in a ecificawould be to be a finite or the party of the tions.

F. Governing Design Loads - From the point of view of design-If the he may like the horsest series of I be provided to be large ing or selecting a wood member adequate to resist a desi n load of given ma nitude, the required cross-sectional property of the sember is a function of the total does n load divided by the allowable unit stress. This is true irrestective of whether the stress function is a bending stress, an axial stress or a shear stress. For example

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this relation may be expressed for the cases mentioned as follows:

$$S = \frac{M}{f}$$
 $A = \frac{P}{t}$ (for axial tension) $A = \frac{3V}{2h}$ (for rectangular beams)

in which the required cross-sectional properties of the member are S, the section modulus and A, the cross-sectional area; the imposed that he provide Date had not been in factor to desirn louds are M, bending moment, !, tensile load and V, shear; and unit working stresses are f, allowable stress in extreme fiber Married III I commission that it was due to bending, t, allowable tensile stress, and B, allowable horizontal shear stress. Wow taking the eneral can where X is the required cross-sectional property, I is the total design load and u is the allowable unit stress, the relation is expressed thus:

$$x = \frac{n}{u}$$

 $x = \frac{D}{u}$ Let Doi, Dil, Dy, and DI represent the maximum design loads for dead load, live load, wind and impact respectively. Then according to the various permissible increases of u for the different types of Della consideration of the property of the party of the p losde we have:

$$x_1 = \frac{D_{DL}}{u}$$
 $x_2 = \frac{D_{DL} + D_{LL} + D_{W}}{1.50 \text{ u}}$ $x_3' = \frac{D_{DL} + D_{LL} + D_{W}}{2 \text{ u}}$

and the required X is the largest of the three. These expr seions may be rewritten as:

$$x_1 = \frac{D_{DL}}{u}$$
 $x_2 = \frac{2/3}{3} (\frac{D_{DL} + D_{LL} + D_{R}}{u})$ $x_3 = \frac{1}{2} (\frac{D_{DL} + D_{LL} + D_{R}}{u} + \frac{D_{L}}{u})$

It can be seen that the largest value of X is governed by the lar est value of the three expressions DDL, 2/3 (DDL + DLL + D) and 1/2 (DoL + DLL + Dy + DI). Fow let us exa ine the rel tive ma nitudes of these three composite loads.

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Then
$$50_{DL} > 2/3(D_{DL} + D_{LL} + D_{W})$$

Also $D_{DL} > 2D_{LL} + 2D_{W}$

This states that in order for T_L to be reater than $2/3(D_{TL} + D_{LL} + D_{LL}$

Next let us compare $2/3(D_{DL} + D_{LL} + D_{N})$ with $1/2(D_{DL} + D_{LL} + D_{N} + D_{I})$.

Assume $2/3(D_{DL} + D_{LL} + D_{N}) > 1/2(D_{DL} + D_{LL} + D_{N} + D_{I})$ Since $D_{I} = 0.3 D_{LL}$

Then
$$2/3(D_L + D_L + D_W) > 1/2(D_{DL} + D_{LL} + D_W + 0.3D_{LL})$$

 $4(D_L + D_{LL} + D_W) > 3(D_{DL} + 1.3D_{LL} + D_W)$
 $4D_{DL} + 4D_{LL} + 4D_W > 3D_{DL} + 3.9D_{LL} + 3D_W$
 $4D_{DL} + 0.1D_{LL} + D_W > 0$

This states that for $2/3(D_{DL} + D_{LL} + D_{L})$ to be greater than $1/2(D_{DL} + D_{LL} + D_{L})$ the sum of D_{DL} , D_{N} , and $0.1D_{DL}$ must be greater than zero. This obviously will always be true again even if D_{L} is neglected. Therefore X_2 will be greater than X_3 and will be the greatest of the three.

The conclusion is that the design of wood members can be based on a hypothetical design load of two thirds of the sum of dead load, live load, and wind load using the allowable unit stresses without modification and the resultant structure will be adequate for the loads of various duration.

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IV. DESIGN OF LOOR SYSTEM

A. Decking - In the interest of standardization it would be desirable to have the same deck for all the bridges under investigation. The design of the decking will therefore be effected with this objective in mind.

The deek will consist of two layers of lumber; the bottom layer is the deck proper which provides the structural resistance to the stress s project by the traffic loads and the top la or is the wearing course whose primary function is to protect the deck from dama e which might be inflicted by the using traffic. The wearing course is a naidered especially necessary in military brides because of the relatively hi h incidence of tracked vehicles emon- the using traffic which incur unusually severe wear on deck surfeces. The wearing ocurse incidentally helps to distribute the wheel loads longitudinally to the deck proper when favorably crinted but exactly to what extent the distribution is enhanced in a particular arran ement is difficult to determine. If the planks of the wearing course are priented longitudinally the load distribution will be improved to the gratest extent. At the same time such an arrangement incurs a hazard should one of the planks become loosened under the action of traffic and bend up above the floor surface. If the planks are placed diagonally across the rosiway, the load distribution is decreased so swhat but probable takes to the flooring resulting from a loose clank will also be reduced. For this reason the latter arrangement is deemed more desirable.

The deck proper may be constructed in several different ways.

the latest section (1)

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party formatted between it have been all the plants and said

Three commonly used types of all-lumber construction are the plank dock, the laminated dock and a look consisting of tengue and groove or a lined or some other well-doweled fabrication.

tributing the applied wheel load longitudinally more effectively than the other types. However the use of such a deck in military bridges is not considered practical for the following reasons. The enlines or the tengues would not stand up under the usual handling which occurs in getting lumber materials from the mill to the site of military operations. In order to be effective the joining fit between planks must be near perfect and such practices as open storage in the combat zone might produce either swelling or shrinkage to such an extent as to preclude this. Such materials also require more care in placing and therefore take longer to put down. Jonesquently this type deck will not be considered further.

The laminated deck, which consists of narrow planks laid on edge without interval, has the advantage of combining fair lead distribution with the required structural strength for heavy wheel leads. The wheel lead is commonly assumed to be distributed over a width of 15 inches in the direction of travel when the laminated deck is overlain with a flexible wearing course. Taking into consideration the stiffness of a timber wearing course, increasing this distribution by one third to a width of 20 inches seems justified. From a military point of view the leminated deck has the disadvantages of requiring too long to place and presenting a solid surface which does not permit sufficient drainage of the deck.

The second secon

The plank deck, which has been used extensively in military bridges in the pest, appears to offer the most suitable compromise in the various considerations of structural strength, load distribution, drainers, speed in placement and ability to withstand rough handling. The limiting feature of the plank dock is the longitudinal distribution of the wheel lond. The usual assumption is that with a flexible wearing course the entire wheel load is distributed longitudinally over the width of only one plank. However it does not appear unreasonable to assume, in the case of a superimposed timber wearing course laid dis enally, that the full wheel load may be sensitored as distributed over the width of two planks. A unique adventage is found in the plank dock with regard to drainage. Since its load resisting capacity does not domand the direct contact of adjacent planks, the planks of the dock proper as well as those of the wearing course can be laid with a small intervening space to allow almost immediate escape of rain water. The prevention of pending on the floor surface is rather important because saturation of the wood decreases its strength.

Up to this point it is concluded that the flooring will consist of a timber wearing course laid diagonally and a dock proper of either a leminated dock or a plank dock, whichever is most advantageous from the overall point of view.

Design of the dock entails the selection of a dock section and determining the maximum effective s an length over which that particular dock will safely support the design wheel lead. Jubsequently the stringers are erran ed in such a manner as not to

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considered adequate. The effective sum length of the dech is then considered adequate. The effective sum length of the dech is the center to center to center a acid of the stranger length of the it of one stringer. In computing the bending and the centionalty of the deck planks is introduced. The maximum applied moment is assumed to be eight—tenths of the maximum paent if the deck were climated as a simple beam between supports. On lateral distribution of the wheel load to adjocent deck spans is taken into excount. In computing shear the usual practice of impring all lock within one plank's depth of the theoretical support is also applied.

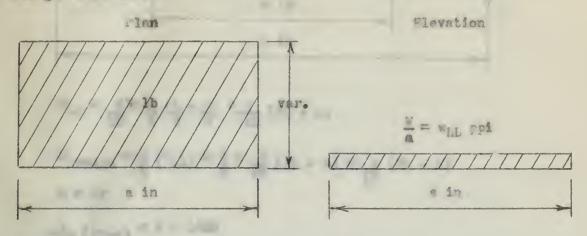
should be rather wide to provide a substantial count of structural stringth as well as to enhance clacin efficiency by providing a large deck surface area per individual niece handled.

The death must be sufficient to provide the tructural structural structural etror—th necessary to permit reasonable stringer sections. But above all the clank selected must be commonly available for procure ent from the destic lumber industry in quantity. A 3" by 12" or 4" by 12" plank fits these re-uirements fairly well and each will be used for a trial plank deck. Ith the same considerations in find, a trial laminated deck will consist of 2" by 4" strips on edge.

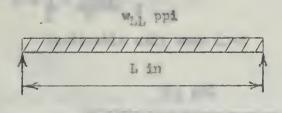
DECK DESIEN

Allowable Unit Stress - f = 1600 psi E = 120 psi E = 1,600,000 psi Limiting Deflection - 1/200 of span Assume dead load to be negligible.

Design theel Load -



Flemure -



$$M_{\text{Design}} = \frac{2}{3} (M_{\text{LL}}) = \frac{2}{3} \times \frac{W_{\text{LL}}L^{0}}{10} = \frac{W_{\text{LL}}L^{0}}{15}$$

$$M = 80^{\circ}$$
 $\frac{W_{LL}^{LS}}{15} = 8 \times 1600^{\circ}$

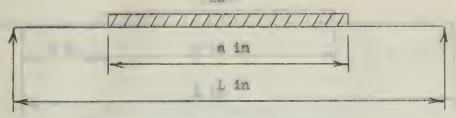
$$L = \sqrt{\frac{2400}{w_{LL}}} s$$

Applicable only when L< a

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Flexure (continued)

M.I. Phy



$$M_{LL} = \frac{8}{10} \times \frac{4}{2} \left(\frac{L}{2} - \frac{a}{4}\right) = \frac{4}{10} (2L - a)$$

Moesign =
$$\frac{2}{5}$$
 (MLL) = $\frac{2}{3}$ × $\frac{4}{10}$ (2L - a) = $\frac{4}{15}$ (2L - a)

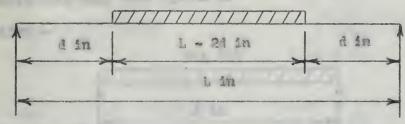
$$\frac{1}{15} (2L-a) = 8 \times 1600$$

$$L = a + 12000$$
3

Applicable only when L>a

Shear -

W- 1. 1 P2



$$A^{\Gamma\Gamma} = A^{\Gamma\Gamma}(\frac{3}{\Gamma-5q})$$

$$V_{\text{Design}} = \frac{2}{3} \left(V_{\text{LL}} \right) = \frac{2}{3} \times W_{\text{LL}} \left(\frac{L-2d}{2} \right) = W_{\text{LL}} \left(\frac{L-2d}{3} \right)$$

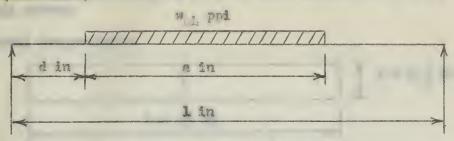
$$M_{LL} \left(\frac{1-24}{3} \right) = \frac{2 \times 4 \times 120}{3}$$

$$I' = 5q + \frac{M^{17}}{5p0} \quad V$$

Applicable only when L <a + 2d

P + # 1600 4 >

Shear (Continued)



$$V_{LL} = \frac{1}{L} \left(L - d - \frac{R}{2} \right)$$

$$V_{\text{Desi}} = \frac{2}{3} (V_{LL}) = \frac{2}{3} \times \frac{W}{L} (L - d - \frac{a}{2})$$

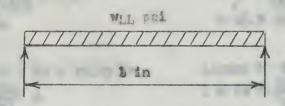
$$V = \frac{2AH}{3}$$

$$\frac{2^{1}}{3L}$$
 (L - d - a) = $\frac{2 \times A \times 120}{3}$

$$L = \frac{\left(d + \frac{a}{2}\right)}{-120}$$

Applicable only when L>a + 2d

Deflection -



$$\Delta = \frac{3}{384} \frac{\text{MLL}^{4}}{11}$$

$$\frac{L}{200} = \frac{5}{304} \times 1,600,000 \times 1$$

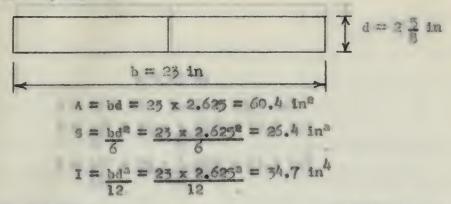
$$L = \frac{3}{4} \frac{614400}{11}$$

Applicable when L < a; wen L > a, result is conservative and unless deflection is critical will be a sufficient cless.

28.3 10-1-27-5 - n - 10 5" b" un" b" mises? ENTHRY <

Try Plank Deck consisting of 5" x 12" deck proper and 2" x 12" wearing course.

Sectional Properties -



LITHT RIDGE

PECTURE YVALUE

Design Wheel Load -
$$M = 16000 \text{ lb}$$
 $M = 20 \text{ in}$ $M_{LL} = \frac{M}{M} = \frac{16000}{20} = 800 \text{ ppi}$

$$W = 24000 \text{ lb}$$
 $a = 30 \text{ in}$ $W_{LL} = \frac{W}{a} = \frac{24000}{30} = 800 \text{ ps} 1$

Flower -
Assume L a = 20 in
L =
$$\frac{a}{2} + \frac{12000}{a}$$
 s
= $\frac{20}{2} + \frac{12000}{15000} \times 25.4$
= 29.5 in

Assume L
$$a = 50$$
 in
 $L = \sqrt{\frac{24000}{W_{LL}}}$ S
 $= \sqrt{\frac{24000}{800}} \times 26.4$
 $= 38.2$ in

Shear -
Ansume L a + 2d = 25.25 in
L = 2d +
$$\frac{240}{\text{WLL}}$$
 A
= 2 x 2.625 * $\frac{240}{800}$ x 60.4
= 25.4 00VERNS!

Assume L a + 2d = 55.25 in

L = 2d +
$$\frac{240}{W_{LL}}$$
 A

= 2 x 2.625 + $\frac{240}{800}$ x 60.4

= 25.4 in GOVENS!

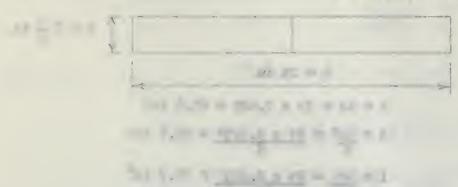
Deflection -

$$L = \frac{3}{614400} I$$

$$= \frac{3}{614400} \times \frac{34.7}{800} \times \frac{34.7}{800} = 29.8 in 0%$$

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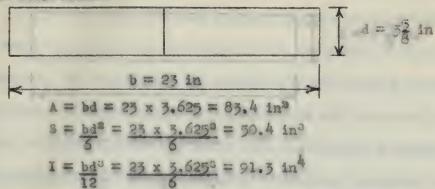
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THE PARTY OF THE P

Try rlank Dock consisting of 4" x 12" dock proper and 2" x 12" wearing course.

Sectional Properties -



LIGHT PRIDGE

Design heel Load -= 16000 lb a = 20 in $w_{LL} = \frac{v}{a} = \frac{16000}{20} = 800 \text{ ppi}$

Flexure -

Assume L a = 20 in

$$L = \frac{8}{2} + \frac{12000}{V}$$
 s

 $= \frac{20}{2} + \frac{12000}{16000} \times 50.4$
 $= 47.8$ in

Shear -

Assume L n + 2d = 27.25 in
L =
$$V(d+\frac{a}{2})$$

 $V = 120A$
= $16000(3.625 + \frac{20}{2})$
 $16000 - 120 \times 83.4$
= 36.3 in $000^{-1} \cdot 3$!

HEAVY B IDE

$$W = 24000 \text{ lb } a = 30 \text{ ln}$$
 $W_{1L} = 0 = 24000 = 800 \text{ pp1}$

Assume L a =
$$\frac{30}{2}$$
 in
L = $\frac{a}{2} + \frac{12000}{V}$ s
= $\frac{30}{2} + \frac{12000}{24000} \times 50.4$
= 40.2 in

Assume L a + 2d = 57.25 in
$$L = 2d + \frac{240}{w_{LL}} a$$

$$= 2 \times 3.625 + \frac{240}{600} \times 83.4$$

$$= 32.2 \text{ in } 900 \times 83.4$$

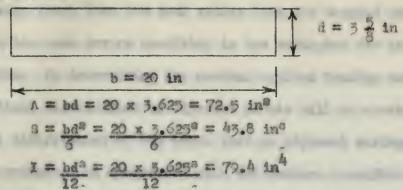
Deflection -
$$L = \sqrt[3]{\frac{614400}{W_{LL}}} = \sqrt[3]{\frac{614400}{800}} \times 91.3$$

$$= 41.2 \text{ in } 0.8$$

the state of the second state of the substitution and such such sections. - MANAGEMENT AND ASSESSED. THE DATE OF THE REAL PROPERTY OF THE to be when the plant of the last Mark June A. In 1 (1987) * 15 ** - 1 0.7° -£ 44 = I 1. 17 to to. 1 = 1 = - F2 '5 700 T = 7-10 m - 10 m 1 - 1,00 -

Try Leminsted Deck consisting of 2" x 4" on edge with 2" x 12" wearing course.

Sectional Properties -



LIGHT BRIDGE

Design Wheel Lond -
$$W = 16000 \text{ lb}$$
 $A = 20 \text{ in}$ $W_{LL} = \frac{U}{2} = \frac{16000}{20} = 800 \text{ ppi}$

Flexure -

Accumo L a = 20 in

L =
$$\frac{a}{2} + \frac{12000}{\pi}$$
 S

= $\frac{20}{2} + \frac{12000}{16000} \times 43.8$

= $\frac{42.8}{2}$ in

Shear -

Assume L a + 2d = 27.25 in
L =
$$\frac{1}{4} \cdot \frac{1}{2}$$

 $\frac{1}{4} \cdot \frac{1}{20}$
= $\frac{16000}{16000} \cdot \frac{3.625}{12.5} + \frac{20}{2.5}$
= 29.9 in GOVERNS !

HIAVY BRIYE

Assume L a = 30 in

$$L = \frac{a}{2} + \frac{12000}{8}$$
 s
= $\frac{30}{2} + \frac{12000}{24000} \times 45.8$

W = 24000 1b a = 30 in

Absume L a + 2d = 37.25 in

L = 2d +
$$\frac{240}{W_{LL}}$$
 A

= 2 x 3.625 + $\frac{240}{800}$ x 72.5

= 29.0 in dovums!

Deflection -

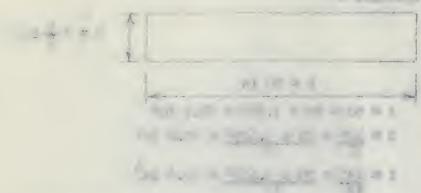
L =
$$\frac{5}{614400}$$
 I

= $\frac{3}{614400} \times 79.4$

= $\frac{3}{600} \times 79.4$

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SENIOR STANFO

B. Stringers - ith the maximum effective span len th of the trial docks determined the stringers can now be deal med with this limitation in view. It is apparent from an in paction of the alternate design leads that the tank rather than the whooled vehicle will imose the more severe condition in the stringers for the usual panel lengths. In determining the maximum applied bending moments no longitudinal distribution to adjacent panels will be considered but lateral distribution of the track load to adjacent atringers will be ap roximated in accordance with the factors secified in the A.A.J. . specifications. The fraction of the load used to calculate the bending moment is L where L is the stringer spacing in feet and I is a constant depending on the number of traffic lanes and the type of deck. For a single-lane bridge 3 is 4.00 for a plank dock and 4.50 for a 4-inch laminated dock; for a doublelane bridge ? is 3.75 for a plank deck and 4.00 for a 4-inch laminated deck. These fractions are considered appropriate even though they are specifically applicable to concentrated which loads whereas a uniformly distributed track load is being dealt with in the case at hand. The fractions contained in the A.A.S. 1.O. specifications were in all probability derived empirically for a single concentrated wheel load at mid span, for that is the position in which the load would be placed to compute the maximum bending moment. The fraction merely reflects the fact that as the stringer under the concentrated load deflects and the deck also deflects, the stringer is relieved of a portion of the load through the action of the deck in resisting the deflection. In other words aportion

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of the concentrated load is laterally distributed to edjacent strin ers by virtue of the stiffness of the doc'. Tow as the concentrated load is moved away from the center of the span toward the end of the stringer, the deflection decreases and therefore the shility of the deck to distribute the lond laterally is not fully used. For example at the quarter point the relief due to lateral distribution is approximately eighty per cent of that at the mid point. So, for a uniformly distributed load it is slightly insecurate to reduce the intencity of load throughout its entire length on the basis of the reduction applicable only at mid spen. But it is felt that this is adequately compensated for by the fact that no analytical consideration is taken of the stiffness of the wearing course which in effect improves the lateral distribution at all points of the stringer span. In computing borizontal shear in the stringers the same degree of lateral distribution will be considered effective; however no lead within one stringer's death of the theoretical support will be essociated with shear at the neutral axis.

From the foregoing discussion regarding lateral distribution it is seen that the size of the stringer required to support a particular load will vary with the stringer spacing. The stringer spacing may be varied at will between a prectical minimum and the maximum effective deck span. With standardization in mind it would be desirable to have the stringers for the various bridge structures all the same size. This may possibly be accomplished by using near maximum stringer spacing for the light bridge and the same stringer at a closer spacing for the heavy bridge. Such will be attempted

The second secon The second of th the same of the sa the state of the second st THE RESERVE AND ADDRESS OF THE PARTY OF THE the second property of the facility of the second section of the party of THE RESIDENCE OF THE PARTY OF T the second of th the state of the s at the same of the AND RESIDENCE OF THE PARTY AND PARTY per la company de la company d the second of the second second second second The second state of the se bed to be a produced by the party of the par to Title brimes Statistically all Vo. Shell of passages are SAMPLE makes I realized with the reason of the Dark Statement

in the subsequent design.

Also with recard to stringer opscing it is probable that the stringers in one and will 'av to be offset later lly from strin era of adjacent panels. This is necessary because the face of the amporting caber, either a floor beam or a bent cap, will in all robability not be wide enough to provide ufficient bearing area for stringers placed and to and. On the other and the curb blocks at either side of the clear roadway, which must be coling r for the entire lengt of the structure, will no doubt be bolted through to the outside stringers. Consequently the outside stringers at be in line end to end des ite the limited a rin ar a. This situation is not considered serious because the outside stringers ere not subject to the loads that the interior stripers must withstand due to the fort that traf is loads cannot be au erim osed directly over them and yet for the sake of unifor ity they will be the same size. It is therefore concluded that the striper sacing, b, in a particular pool will be constant to left to right with the exception of the right end e ace shich will be L less one strin er's broadth. Then in adjacent panels the interior stringers will be shifted loft one stringer's broadth resulting in a contant spacinfrom right to left except for the left end space which will main be L les one trin er's breadth.

In selecting the actual stringer section the consideration of lateral booking of the compression for the trial into account.

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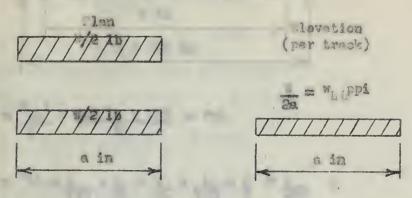
the beam but by stipulating the limiting depth to breadth ratio of the beam for various degrees of lateral support ascorded the compression face. If the ratio of depth to breadth is 2 or less to 1, no lateral support is required. If the ratio is between 2 and 3 to 1, the ends of the beam must be positively held in place. For greater ratios of depth to breadth more elaborate lateral support is prescribed. In order to avoid lateral support of stringers altegether and attendant inclusion in the floor design of devices necessary to provide such support, stringers with a depth to breafth ratio no reater than 2 to 1 will be used if practicable.

The panel length is assumed to be fifteen feet. This will
permit the procure ent and use without cutting of sixteen-foot
atrimers which is a commercially eveileble length. Treater panel
lengths will entail proportionately larger and longer stringers
and it is feared that stringer size timbers of over sixteen feet
in length may be difficult to obtain in quantity. From a logistical point of view it would be difficult if not impossible to determine the most economical panel length because of the many
variables involved. For these reasons fifteen feet has been selected as the upper limit of practical panel length for the preposed design. Furthermore this selection will permit use of leaser
and lengths with the determined stringers without any danger
should the situation domand it.

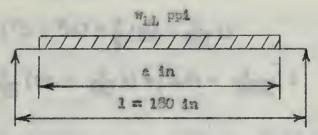
STAINGER DESIGN

Allowable Unit Stress - f = 1600 psi H = 120 psi E = 1,600,000 psi Assumo 15-foot panels.

Besign Load -



Flexure -



Let 1 = center to center specing of stringers in inches Let 0 = constant for determining lateral distribution fraction

$$w_{DL}$$
 (for deck) = $\frac{6}{12} \times \frac{1}{12} \times \frac{1}{12} \times 40 = 0.159L$

= 0.139L (satimated same as for deck)

= 0.278L

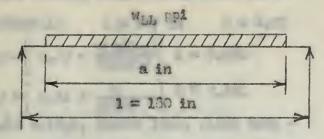
$$N_{\rm DL} = \frac{v_{\rm DL} 1^{3}}{8} = \frac{0.7781 \times 180^{9}}{8} = 1127L$$

$$N_{LL} = \frac{1}{2} \times \frac{1}{120} \times \frac{1}{2} (\frac{1}{2} - \frac{a}{4}) = \frac{1020}{1920} L$$

$$M_{\text{Design}} = \frac{2}{3} (M_{\text{DL}} + M_{\text{LL}}) = \begin{bmatrix} 751 + \frac{11}{2000} \\ 2000 \end{bmatrix} L$$

$$6 = \frac{1}{2} = \frac{1}{1600} \left[751 + \frac{\sqrt{(360-a)}}{2630} \right] L = \left[0.47 + \frac{\sqrt{(360-a)}}{4610000} \right] L$$

ALMERICAN DESCRIPTION OF THE PERSON OF THE P LINE WILLIAMS 72 21 48 The second of th money of the farmers - Marin - Negation | West | Same The second of th Shear -



$$V_{DL} = \frac{w_{DL}^1}{2} = 0.275L \times 180 = 25L$$

$$V_{LL} = {}^{M}LL \times {}_{L} \times {}_{R} \times {}_{R} = {}_{M} \times {}_{L} \times {}_{R} \times {}_{R} = {}_{M} \times {}_{L} \times {}_{R} \times {}_{R} = {}_{M} \times {}_{R} \times$$

$$V_{\rm Besign} = \frac{2}{3} (V_{\rm DL} + V_{\rm LL}) = \frac{2}{3} (29L + \frac{4}{400} L)$$

27.2

$$A = \frac{7V}{20} = \frac{1}{120} (25L + \frac{V}{480} L) = \left[0.21 + \frac{V}{57600}\right] 1$$

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March 2 September 201 A P. College Street

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For Light Bridge, single lane, plank deck -

$$W = 74000 \text{ lb}$$
 $a = 147 \text{ in}$ $0 = 4.00$

Red $S = \begin{bmatrix} 0.47 + \frac{360-a}{4610000} \end{bmatrix}$ $L = 9.02L$

Red $A = \begin{bmatrix} 0.21 + \frac{44}{57600} \end{bmatrix}$ $L = 3.42L$

For Light Tridge, double lane, plank dock -

$$V = 74000 \text{ lb}$$
 $a = 147 \text{ in}$ $0 = 5.75$

Red S =
$$\left[0.47 + \frac{11(360-a)}{46100000}\right]$$
 L = 9.59L

Rad A =
$$\left[0.21 + \frac{11}{57600}\right]$$
 L = 3.64L

For sake of standardization let elightly larger requirements of double lane bridge govern for both structures.

Required clear roadways: Single Lane - 150 in and Double Lane - 254 in Assume curb blocks to be 8 inches wide.

| Triel | Minisum Stringer | Required | Required | Trial String | er Pessible | Spacing |
|---------|---------------------|----------|-----------------------------------|--------------------------|--|---------------------|
| Spacing | Broadth | Modulus | | Size | Single Lone | Double Lene |
| L | b=2(1-35.3) | ≈9.59L | 4=3.64% | bxd | n at l=nl+L-l=158 | |
| 41 | 10 | 393.19 | 149.24 | 10x18 | 5at41=125+31=154 | 6at41=246+51=277 |
| 40 | 8 | 363.60 | 145.60 | 10x18 | 3at40=120+50=150 | 6at40=240+30=270 |
| 30 | 6 | 374.01 | 141.96 | 10x16 | 3at37=117+29=146 | 6e.t 39=23/H 29=263 |
| 33 | 4 | 364.42 | 158.52 | 10x16 | 4at;38=150+29=180 | 6at 36=228+28=256 |
| 57 | 2 | 354.63 | 154.68 | 10×16 | 4et5/=148+27=175 | 7at5/=299+27=288 |
| 36 | - | 345.24 | 151.04 | 10x16 | 4at36=144+26=170 | 7at36=252+26=278 |
| 35 | - | 335.65 | 127.40 | 10×16 | 4at55=140+23=165 | 7at39=245+29=270 |
| 34 | | 326.06 | 125.76 | 10x16 | 4at34=136+24=160 | 7at54=258+24=262 |
| 55 | - | 316.47 | 120.12 | 10x16 | 4at35=132+25=155 | 8at35=264+23=287 |
| 52 | - | 305.88 | 116.48 | 10x16 | hat32=120+22=150 | 8at32=256+22=278 |
| 31 | • | 297.29 | 112.84 | 8x16 | 4at31=124+23=147 | 8at31=248+27=271 |
| 30 | - | 287.70 | 109.20 | 8x16 | 5et30=150+22=172 | Set 30=2/0+23=262 |
| 29 | 300 | 278.11 | 103.56 | 8x16 | 5at29=145+21=166 | 9at29=261+21=282 |
| 28 | - | 268.52 | 101.92 | 8x16 | Set 38=140+20=160 | 9at28=252+20=272 |
| 27 | - | 258.93 | 98.28 10x18 (3=1 10x16 (3=1 | 8x16 184.90 280.40 | $5at27=155\cdot19=154$ and $A = 166.25$) and $A = 147.25$) | 9at27=243+19=262 |

8x16 (\$=300.31 and A = 116.25)

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| | A STATE OF THE PARTY OF | 45 | | 10,000 | 94 | W |

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$$R = 110000 \text{ lb}$$
 $a = 152 \text{ in}$ $0 = 4.00$
 $R = 0.47 + \frac{1(360-a)}{4610000} L = 12.87L$

Rqd A =
$$\begin{bmatrix} 0.21 + \frac{4}{57603} \end{bmatrix}$$
 L = 4.98L

For Heavy bridge, double lane, plank deck -

$$W = 110000 \text{ lb}$$
 $a = 152 \text{ in}$ $0 = 3.75$

Rqd
$$S = \left[0.17 + \frac{16(360-a)}{46100000}\right] L = 15.70L$$

Rqd A =
$$\left[0.21 + \frac{W}{57600}\right]$$
 L = 5.30L

For sale of standardization let slightly larger requirements of double lane bridge govern for both structures.

Required clear readways: Single Lane - 150 in and Double Lane - 264 in Accuracy curb blocks to be 8 inches wide.

| Trial Stringer Spacing | Stringer Breadth | Required Section Modulus | Required Area | Trial Stringe Size | er Posnible Single lane | Spacing Double Lane |
|------------------------------|---------------------|--------------------------------|------------------|--------------------------|----------------------------|------------------------|
| L | 1=2(1-32.2) | 3=13.70L | 1=5.30h | bxd | n t 1=1+1-1=194 | national-6=344 |
| 31 | | 424.70 | 164.30 | 10x18 | 6at31=186+21=207 | 10et51=310+21=351 |
| 30 | | 411.00 | 159.00 | 10x18 | 6at 30=180+20=200 | llat50=550+20=550 |
| 29 | - | 397.30 | 153.70 | 10x18 | 6at29=174+19=193 | 11at29=319+19=358 |
| 23 | 89 | 533.60 | 148.40 | 10x18 | | 12nt28=336+18=354 |
| 27 | | 359.90 | 143.10 | 10x16 | | 12at27=324+17=341 |
| 26 | - | 356.20 | 137.80 | 10x16 | | 15et26=358+16=354 |
| 25 | 40 | 342.90 | 132.50 | 10x16 | 7at29=175+19=190 | 159129=325+19=340 |
| 24 | | 328.80 | 127.20 | 10x16 | 7at24=168+14=182 | 14at24=356+14=550 |
| 23 | | 315.10 | 121.90 | 10x16 | 8at23=184+13=197 | 14at27=322+17=355 |
| 55 | - | 501.40 | 116.60 | 8x16 | | 15at23=330+14=344 |
| 21 | 400 | 257.70 | 111.50 | 8x16 | | 16et21=356+13=349 |
| 20 | ~ | 274.00 | 106.00 | 8×16 | 9at20=180+13=192 | 17et20=340+12=352 |

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Por Light Bridge, single lane, laminated deck -

$$N = 74000 \text{ lb} \qquad a = 147 \text{ in} \qquad c = 4.50$$

$$Red S = \begin{bmatrix} 0.47 + \frac{8(360-a)}{4610000} \end{bmatrix} L = 8.07L$$

$$Red A = \begin{bmatrix} 0.21 + \frac{8}{57600} \end{bmatrix} L = 3.07L$$

For Light Bridge, double lane, laminated dock -

$$Red A = \begin{bmatrix} 0.47 + \frac{14}{350-a} \end{bmatrix} L = 9.02L$$

$$Red A = \begin{bmatrix} 0.21 + \frac{14}{57502} \end{bmatrix} L = 5.42L$$

For sake of standardization let slightly lerger requirements of double lase bridge govern for both structures.

Required olear readways: Single Lase - 150 in end Rouble Lase - 264 in Assume curb blocks to be 8 inches wide.

| Trial Stringer Spacing | Winimm Stringer Broadth =2(1-29.9) | Modulus | Required Area A=3.421 | Trial String Size b x d | or Possible Single Land nati=n1+L-1=158 | Spacing Double Lane netl=ni+L-b=272 |
|------------------------------|------------------------------------|---------|-----------------------------|----------------------------------|---|-------------------------------------|
| 35 | 8 | 207.66 | 112.86 | 3×16 | | 8at37=364+29=289 |
| 35 32 | 6 | 238.64 | 109.44 | 8x16 | 4at32=128+24=152 | 8at32=256+24=290 |
| 51 | 4 | 279.62 | 105.02 | 8x16 | 4at31=124+23=147 | 8at51=248+23=271 |
| 30 | 2 | 270.60 | 102.50 | 8x16 | 5at30=150+22=172 | 8at 30=240+22=262 |
| 59 | - | 261.58 | 99.18 | 8216 | 5at20=145+21=166 | 9at39=261+21=282 |
| 28 | 40 | 252.56 | 95.76 | 8x16 | 5at20=140+20=160 | 9at28=252+20=272 |
| 27 | | 213.54 | 92.34 | 8x16 | 5at27=135+19=154 | 9at:?7=243+19=262 |
| 26 | en . | 234.52 | 88.92 | 8x16 | 5at26=1 30+18=148 | 10at26=250+18=278 |
| 25 | • | 225.50 | 85.50 | 8x14 | | 10at29=250+17=267 |
| 25 | • | 216.48 | 82.08 | 8x14 | 6at2/=14/+16=160 | 11at24=264+16=290 |

8x16 (S = 300.31 and A = 116.25) 8x14 (S = 227.81 and A = 101.25)

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STRINGER TEST T

For Heavy Wridge, single lane, laminated deck -

$$W = 1100000 \text{ lb}$$
 $a = 152 \text{ in}$ $0 = 4.50$

$$\text{Red 5} = \left[0.47 + \frac{4(369 - 8)}{4610000}\right] L = 11.50L$$

For Heavy Bridge, double lane, laminated dock -

$$W = 1100000 15$$
 a = 152 in $0 = 4.00$

Rqd 3 =
$$0.47 + \frac{N(350-a)}{4610000}$$
 L = 12.37%

Red A =
$$0.21 \div \frac{W}{57000}$$
 L = 4.981

For make of stendardization let slightly larger requirements of double lane bridge govern for both structures.

Re wired clear readways: Single Lane - 150 in and Double Lane - 254 in Amsume curb blocks to be 8 inches wide.

| Triel Stringer Specing | !inimus Stringer !readth | Required Section Fodulus | Required Area | Triel String Size | or Foosible Spacing Bingle Leno Double Lone |
|------------------------------|--------------------------------|--------------------------------|------------------|-------------------------|--|
| L | b=2(L-20.0) | 3=12.87L | 1=4.98L | b x d | notionist-t=194 notionist-t=544 |
| 23 | | 296.01 | 114.54 | 8x16 | 8et27=184×19=199 14et27=322+19=357 |
| 22 | - | 283.14 | 109.56 | 8x16 | 8et22=176+14=190 15et22=330+14=344 |
| 21 | - | 270.27 | 104.58 | 8x16 | 9at21=189+13=202 16at21=336+13=349 |
| 20 | *** | 257.40 | 99.60 | 8x16 | 9et20=180+12=192 16at20=320+12=332 |
| 19 | | 244.53 | 94.62 | 8x16 | 10at19=190+11=201 17at19=323+11=334 |
| 18 | - | 231.66 | 89.64 | 8x16 | 10at18=180+10=100 18at18=324+10=534 |
| 17 | - | 218:79 | 84.66 | 8x14 | 11et17=1874 9=196 19et17=323+ 9=532 |

8x16 (8 = 300.51 and A = 116.25)8x14 (8 = 227.81 and A = 101.25)

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It this point it is it be well to tentatively celest the deck as well as the stringer size and corresponding a coin before prooccdin; further with the decime leviewin the trial decks treated earlier, it is a parent that the 3" by 12' plank deck results in a maximum effective door arem which is somewhat low. This would entail the use of a larre number of atringers alosely a good which in turn would unnecessarily increase construction time. . . e 2" by 4" laminated dook offers some improvement in this respect in that it coscares more structural strength and therefore will eafely upon a grouper distance between stringers. Fowever, it 'as the inherent disedvantages, as reviously pointed out, of requiring tedious rlace and and exhibitin boor drains a characteristics. here two disalventeres do not a rear to be outweithed when co paring the 2" by A' la instal dock with the 4" by 12" plank dock. The 4" by 12" plank den' permits even a wiler latitude in the selection of strin or a scin a, which is particularly important if the same string r section is to be used in both the light and heavy bridge. It also el' inates l'e essible difficulties centioned d'rin- construction and mervice.

in examination of the tabulated data partaining to stringer deal m indicates that the laminated dack, because of its creater ability to laterally distribute the load, requires a slightly smaller stringer section than would the plank deck for the same atringer spacing. Towever the difference is not of great importance in the light of the fact that stringers must be selected from a group of compercially available sizes and will not just

The second of the second section is selected in with the second and the second section of the second sections and the same of th - The state of the same of the same and the 1) IN COLUMN THE PROPERTY OF THE PARTY OF TH The Control of the Co to be at heart the property we serve on breaded I THE RESIDENCE OF THE PERSON NAMED IN COLUMN 2 IN THE PERSON NAMED IN THE the second of th which will be produced by the production of the party of the last of the same of the last of the la AND RESIDENCE OF A PARTY OF THE The state of the s the state of the parties of the parties and the state of and the second s and the state of t AND THE PARTY SERVICE AND PROPERTY AND EAST AND THE PARTY v

satisfy the demands of the analytical requirements.

Predicated on the desire that the stringer spacing be in even inches for simplicity in construction and that the required clear widths of roadway be adhered to as closely as possible, the use of $8^n \times 16^n$ stringers at 28-inch spacing for the light bridge and 22-inch spacing for the heavy bridge provides a workable solution. Though selected for use in emjunction with $4^n \times 12^n$ plant deck the above arrange ent will take a $2^n \times 4^n$ laminated deck nicely with only a slight margin of over-design if streumstances in the field should necessitate.

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O. Floor-beams - In recognition of the many variables which affect the design of a treatle bent, it is questionable whether a ringle standardized design could be devised to meet the requirements of any site which might be encountered. The wide range of bent heighths which must be anticipated indicates that a variety of large timbers must be provided and used according to the demands of the altustion at hand. And that is the current practice. Admittedly a therough investigation could produce some improvement but whether, from a logistical point of view, it would result in a substantial simplification of the situation is problematical. Hence an extensive treatment of treatle bent design will be dispensed with here and the matter of floor-beam design for truss atmotures will be undertaken.

the floor-beens will be designed in the usual manner as beens simply supported at either end. They are to support the appropriate floor system, as tentatively selected in the provious section, with the proper design tank for the live lost. The truss center lines will be taken as 2 feet outside of the curb blocks and considered the theoretical points of support of the floor-beams. It is apparent that for the floor-beam spans requisite to the double-lane readways and for the unusually large loads, it will be impractical to provide a single beam to withstand the resulting stresses. Though structurally possible, it is deemed inadvisable from a practical point of view to resort to a trussed beam or more scaplex type of construction for the floor beam. For this reason the design of truss bridges will be limited to those of sin-le-lane width which, for military application, is not at all inexprepriate. Even in the case of the floor-beams for

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the single-lane bridges, a single beam will, in all probability, be unreasonably large in section. The ultimate solution may likely involve the use of more than one beam at each panel point.

The dead load which the been or beams must resist will be taken as the dead weight of the floor system in one panel len th plus the estimated weight of the floor-beam all applied as a uniformly distributed load between points of support. To arrive at an estimated panel length for computing the dead load, it is necessary to look forward a bit to the confi uration of the trusces tiemselves. Let us assume that the trues will be a prallel-cherd Fratt with the height equal to the panel lenth. Further let us assume that all trues members will be fabricated from sixteen-foot len t's of timber. The langest truss ember will be the lis cosls. If the len thof the die onal is sixteen feet that the corresponding renel length will be a grown tely thirteen feet. Therefore a penel learth of thirteen feet will be assumed for use in the floor-bear design. Since the uncut stringers are sixteen feet long they will be used as such and a side lap of one and a half feet at each panel point will corur.

Maximum moment in the floor-beam will be computed with the center of gravity of the design tank directly over and at the center of the beas span. Taximum shear will be determed by placing the tank to one side so that the center of the near track is either three beam's depth from the point of support or at the quarter point whichever is nearest the beam and. If both of these points lie outside the clear readway, the tank will be placed with

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that portion of the dead load within one beam's depth of the theoretical support points will be ignored, which is in accordance with usual timber design procedure.

For Light Truss rite, sincle lene -

Allowable init trees - f = 1600 pei H = 120 pei . = 1,600,000 pei Assume two beams of equal section at each panel point.

Assume 13-foot penels. 'esume wind stresses to be no 14-ibl.

Deal n Load - two concentrated loads of 37000 th outh the inches apart

Beam Span - 216 in

Boad wei ht of one panel -

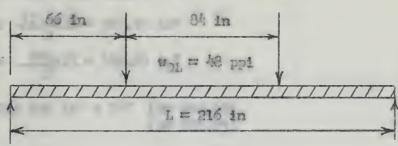
book
$$15 \times \frac{168}{12} \times \frac{6}{12} \times 40 = 3640 \text{ lb}$$

trin rs $8 \times \frac{8}{12} \times \frac{15}{12} \times 16 \times 40 = 4550 \text{ lb}$

Equivalent Uniform Load -

Flexure -

37000 1b 37000 1b



$$M_{\rm DL} = \frac{M_{\rm DL}L^2}{8} = \frac{46 \times 216^2}{8} = 280,000 \text{ in-lb}$$

MLL = 37000 x 66 = 2,442,000 in-1b

 14 testign = $\frac{2}{5}$ (14 DL + 12 L) = $\frac{2}{3}$ (280,000 + 2,442,000) = 1,815,000 in-1b

$$s = \frac{M}{f} = \frac{1815000}{1600} = 1134.50 \text{ in}^3$$

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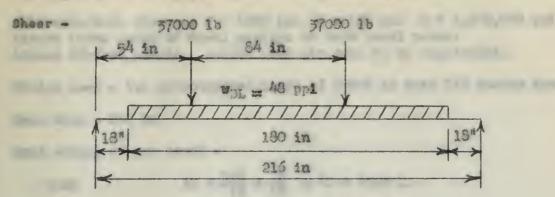
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For Licht Truss ridge, single lene - (continued)



$$V_{\rm DL} = \frac{48 \times 160}{2} = 4520 \text{ lb}$$

$$V_{LL} = \frac{74000 \times 120}{216} = 41110 1b$$

$$V_{\text{Design}} = \frac{2}{3} (V_{\text{DL}} + V_{\text{LL}}) = \frac{2}{3} (4320 + 41110) = 30300 \text{ lb}$$

$$A = \frac{57}{28} = \frac{3 \times 30300}{2 \times 120} = 378.58 \text{ in}^3$$

Required S and A per beam -

Red 3 =
$$\frac{1134.50}{2}$$
 = 557.25 in

Deflection -

$$\Delta = \frac{18500 \times 66 (3 \times 216^{2} - 4 \times 66^{2})}{24 \times 1,600,000 \times 5136.07} + \frac{5 \times 24 \times 216^{4}}{364 \times 1,600,000 \times 5136.07} = 0.84 \text{ an}$$

Permincible
$$\Delta = \frac{1}{200} \times 216 = 1.08$$
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For Heavy Pruss ridge, single lane -

Allowable Unit tress - f = 1600 psi H = 120 si = 1,600,000 psi Assume three became of equal section at each panel point.

Assume 13-foot panela. Assume wind stresses to be negliable.

Design Load - two gencentrated loads of 55000 1b each 110 inches apart Beam Span - 246 in

Dead weight of one panel -

Dock
$$13 \times \frac{198}{12} \times \frac{6}{12} \times 40 = 4200 \text{ lb}$$

Utringers
$$11 \times \frac{8}{12} \times \frac{16}{12} \times 16 \times 40 = 6260 \text{ lb}$$

Floor-beam
$$3 \times \frac{12}{12} \times \frac{18}{12} \times \frac{346}{12} \times 40 = \frac{3690 \text{ lb}}{14240 \text{ lb}}$$

Equivalent Uniform Lond -

$$u_{DL} = \frac{w_{DL}L^2}{8} = \frac{58 \times 246^3}{8} = 439,000 \text{ in-lb}$$

$$u_{LL} = 55000 \times 68 = 3,740,000 \text{ in-lb}$$

$$M_{\text{Design}} = \frac{2}{3} (M_{\text{DL}} + M_{\text{LL}}) = \frac{2}{3} (459,000 + 5,7\%,000) = 2,786,000 in-1b$$

$$s = \frac{1}{2} = \frac{2785000}{1.00} = 1741.25 in^3$$

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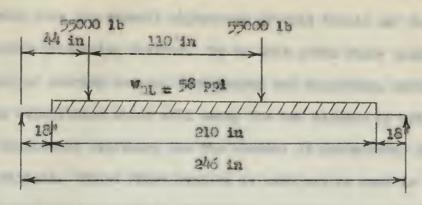
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for Meavy Trues Fridge, single lane - (continued)

Shear -



$$v_{0L} = \frac{98 \times 210}{2} = 6090 \text{ 1b}$$

$$V_{LL} = \frac{110000 \times 147}{246} = 65730 \text{ lb}$$

$$v_{\text{Deelgn}} = \frac{2}{3} (v_{\text{DL}} + v_{\text{LL}}) = \frac{2}{3} (6090 + 65730) = 47900 16$$

$$A = \frac{57}{2N} = \frac{5 \times 47000}{2 \times 120} = 503.75 \text{ in}^3$$

Required S and A per beam -

$$Red A = \frac{298.72}{3} = 199.53 in8$$

Try three
$$12^{4}$$
 x 18^{4} (3 = 586.98) (A = 201.25)

Deflection -

$$\Delta = \frac{18333 \times 63(3 \times 246^{4} - 4 \times 68^{2})}{24 \times 1,600,000 \times 5136.07} + \frac{5 \times 19.33 \times 246^{4}}{334 \times 1,600,000 \times 5136.07} = 1.14 \text{ in}$$

Permissible
$$\triangle = \frac{1}{200} \times 246 = 1.25$$
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V. DESIGN C TUS

A. General - A brief review of the progress thus for and a realignment onto the overall objectives of this thesis may be in order before proceeding further. Up to this point floor systems consisting of wearing owner, deck proper and supporting stringers. have been sutablished which will carry the two classes of options traffic considered lecoscary for the conduct of present-day military operations. These floor systems as such may be used in conjunction with any type bridge structure. Assording to past practice and experience, the floor systems would be intended primarily for use as a co ponent part of a treatle bridge. Jones wently to meet operational needs, it would be necessary to provide a quantity of the timber members which to to make up the floor systems as well as a variety of heavy posts and timbers from which to fabricate treatle bants. A further objective herein is to make the same sizes of wood interfals so provided more versatile in effecting atreem-crossings by devising a scheme whereby trues bridges as well as treatle bridges one be senstmicted from the same ascortment of timber sizes with little if any supplementary meterial required.

Assuming that emong the large members intended for bent fabrication, there are provided 12" x 18" timbers of substantial length,
it has been established that these would suffice handly for floorbeens in the trues bridges. The meterials for the floor systems
consist of 2" by 12"s, 4" by 12"s, and 8" x 16"s. The immediate
problem them is to determine how these sizes can be employed to
construct trueses which will be capable of carrying the two design tents.

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made up from pieces whose length is sixteen feet or less, it

follows that the trusces will be too shallow to sermit the inclusion of everhead bracing. In other words, the use of a conytype trusc is mandatory. With a view toward simplicity in fabricating trusces of various lengths, it would be desirable that
all panels have the same geometric pattern. This indicates the
choice of a parallel-chord trusc over a broken-chord trusc.

In timber trues design, compression members must be designed as columns. Consequently the length of member has a strong influence on the allowable unit strees. On the other hand the allowable unit stress applicable to a tension member is in opendent of its length. Therefore in the case of the web subers, where there is some choice of arrangement, it would be more advantageous to have the short members in compression. The Fratt truss provides this desirable feature. The short vertical was members are primarily in compression and the longer diagonals are in tension. For the same reason, the end penels must be full panels instead of the often used modification wherein the top shord terminates at the lower end panel points. Furthermore the musber of panels should be even in any given truss if counters in the mid panel are to be avoided. As previously mentioned, the limitation on the length of any individual trues member fixes the panel length at approximately thirteen feet. Therefore the variation in span lengths will be in increments of two, and lengths or t enty-six feet.

THE RESIDENCE AND ADDRESS OF THE PARTY OF TH THE PERSON NAMED IN COLUMN TWO PARTY OF TAXABLE PARTY. the fact that the second section is a second section of the sectio the same principle in the state of the state particular and the same say of course of payment and the deal with the facts of presentation to within pass of most president THE RESERVE AS THE RESERVE AND THE RESERVE AND ADDRESS OF THE PARTY AND the Design with the property of the last relating many the comment of the selection of a Deletely we produce Little in the sales like and absolute sales and part are blood all THE RESIDENCE OF THE PARTY OF T the particular of the Lang Law State Service State San State Service States All processors in courts with the sale was all processed to be a processed by the sale of The latest the particular and the latest the the same of the sa The second secon THE RESERVE OF THE PERSON NAMED IN COLUMN 2 IN COLUMN K-1 - 1 - 1 - 1

To sum up, the conclusion thus far is that the truss which gives the most promise of success is a perallel-chord ratt truss with a penal lem th of thirteen feet and a height the same. Various lengths of trusses for the two load-carrying capacities will be investigated some ucing with a four-penal truss and increasing in length two panels at a time to the greatest practical span.

B. Stress in Mombers - Freliminary to attempting the design of any truss members, it might be well to determine in general what the magnitude and range of design stresses are in the various truss members. For this purpose, primary stresses in trueses ereming from 52 to 130 feet for both lead classes will be computed. Dead lead stresses will be determined by applying the dead weight of one-half a floor panel and the estimated weight of one truca panel as a concentrated load at each lower chord panel point. Live load stresses will be calculated under the assumption that only one design tank is on the bridge at one time. It will be positioned laterally with its track flush to the curh block to produce maximum floor beam reaction and longitudinally along the truss so that the stress in the member under consideration is a maximum. In those members subject to reversal of stresses, the counter stresses will else be determined. Jontrary to an earlier statement, wind stresses will not be computed because it is believed that they are of co paratively minor consequence. Impact strasees will be taken as thirty per cent of the maximum live load stresses irrespective of the length span loaded to produce that live load stress.

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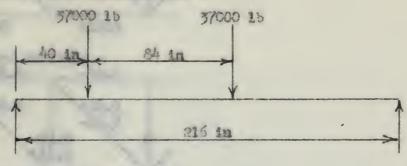
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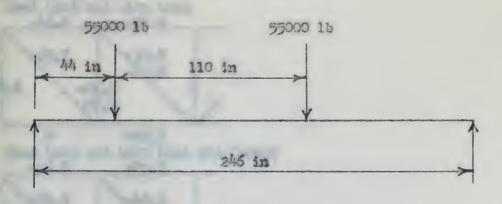


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For Heavy Truss Bridge, single lane -

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Live Load Panel Concentration -



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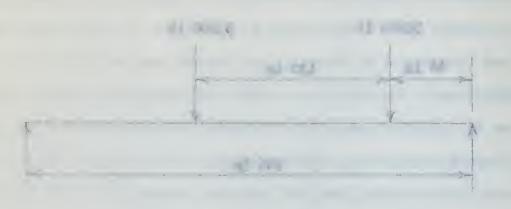
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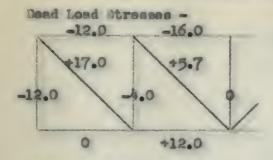
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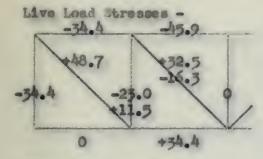


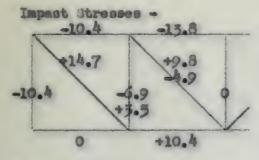
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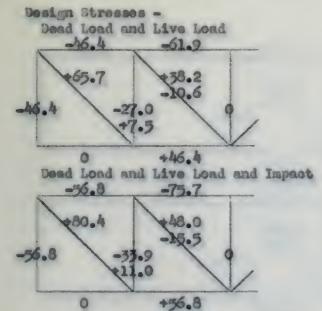
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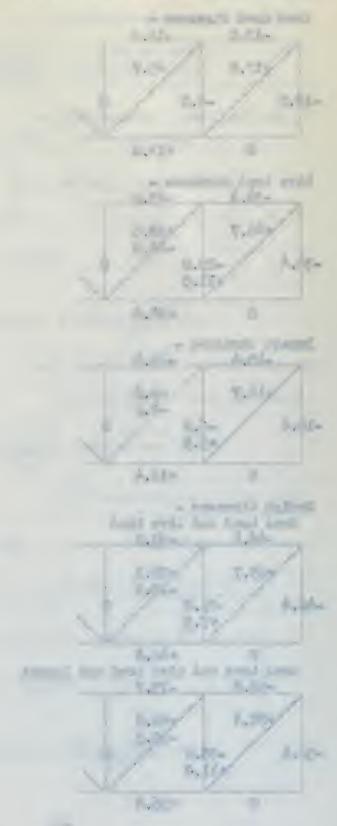






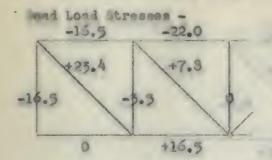
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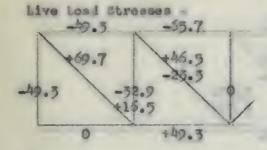
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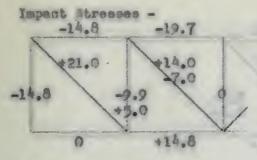


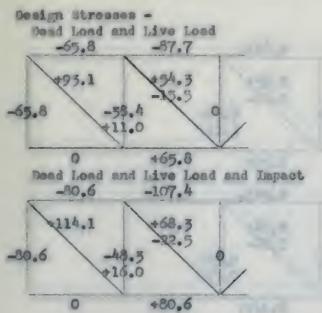
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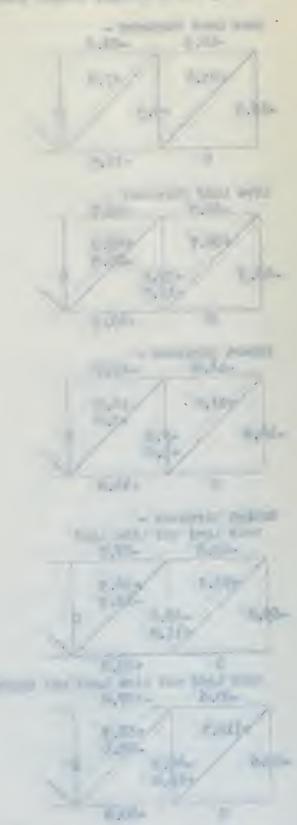






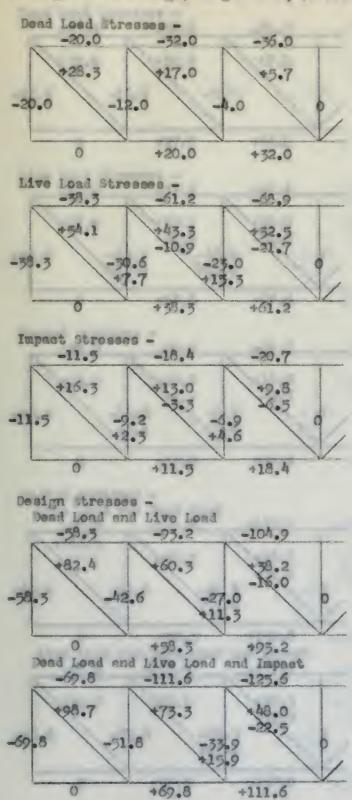
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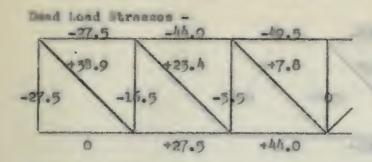
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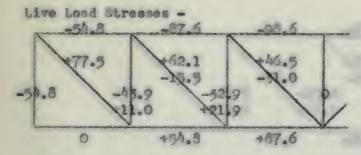
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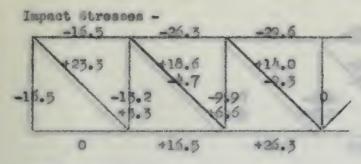


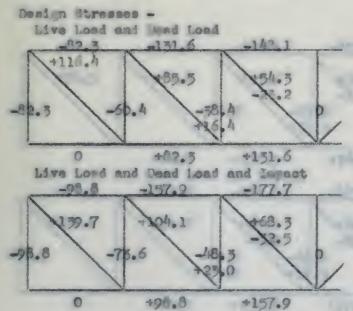
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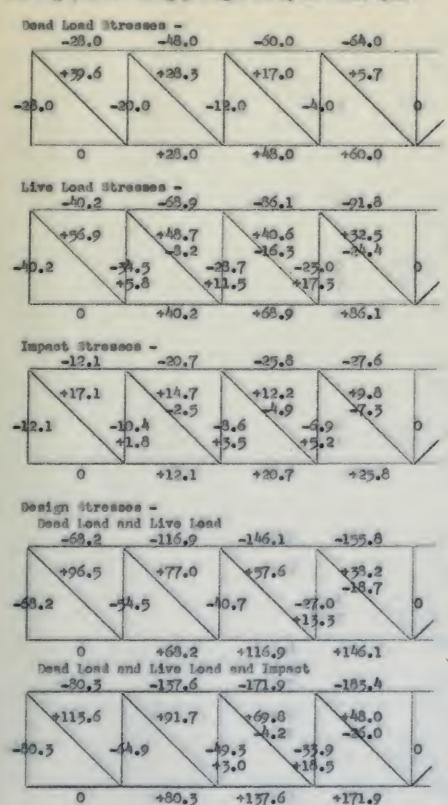




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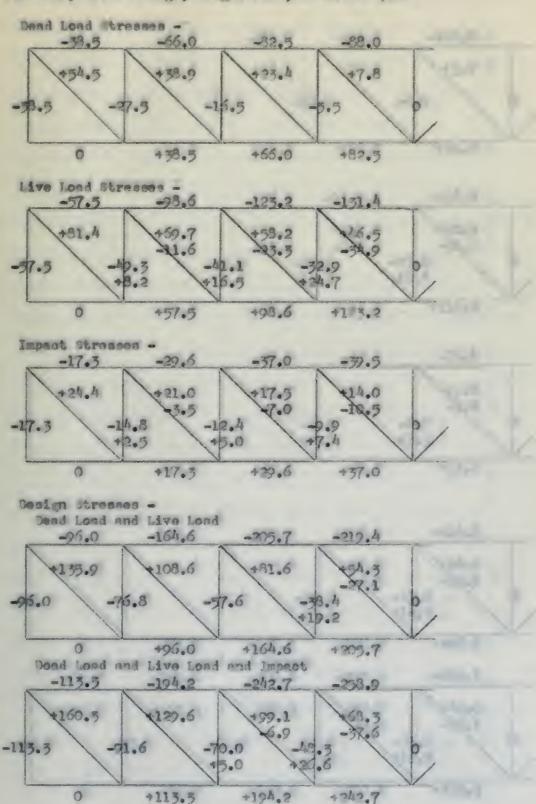
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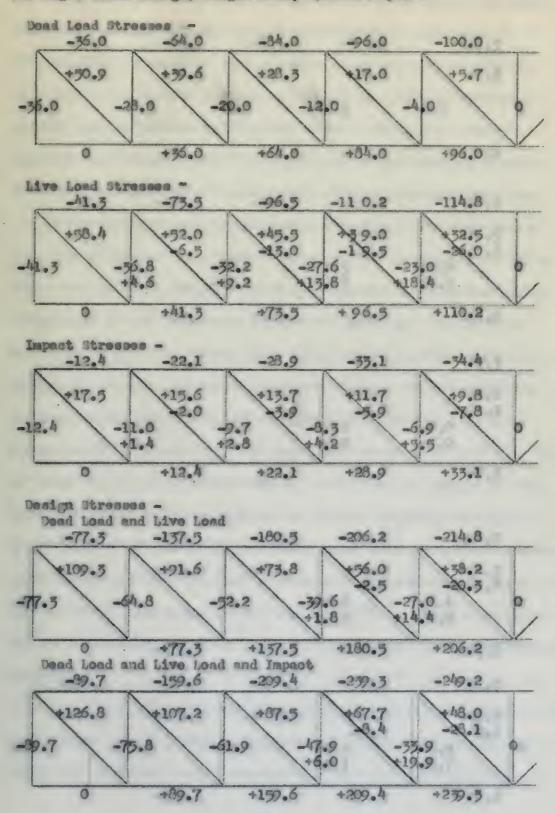
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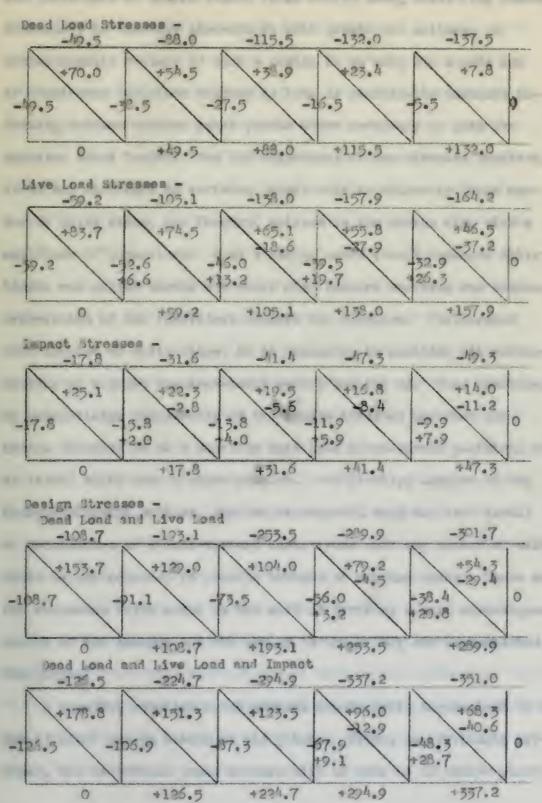
For Light Trues Bridge, single lane, 130-foot span -



THE R. P. LEWIS CO., LANSING COUNTY LAST NAME The sale 1,11 0.25 Sant will T. 11_ -... 1.11-. --CT J+ 7 Real Office the Real Print *t.* 4 2

MEASUR STRESSES

For Heavy Truss Bridge, single lans, 130-foot span -





1. Design of Members - The obvious first thought is to apply the techniques of modern timber truss design using split ring timber connectors to transfer stresses at both joints and splices. A chiracteristic feature of such a design is to make the shords out of continuous one-piece members as long as practically possible inserting splices between panel points where necessary to gain the required total length. For the ma mitudes of the stresses involved, it is not difficult to envision joints with an extremely large number of split rines, and frequent splices in the chords also with a multitude of connectors. Such a design would entail numerous filler blocks and solice blocks and would also require execting and tedious preceration of the individual members for eraction. Furthermore with the use of selit rings, it is hecosary to position all members meeting at a joint simultaneously before bolting up. This practically neconsitates preasonably of the entire truss on the bank and theree swin-in it as a complete unit into cross-stres position, with in its If i bt reas a major roblem. In finally, because of the interpolol-cint splices, such an arrangement does not lend itself to delinection of a well-defined best truss unit eny a ber of which could be not to not to rovide trueses of verying spens. These are the arguments which point up the need of levising a core advantageous don'n of the me bors and the manner in which they can be connected at the joints.

A possible solution which affords considerable improvement is the use of steel cusset plates at the joints. Since, as reviously preeumed, the individual trues — bers will be nade up of the bor places

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no longer than sixteen feet, the steel plates permit the termination of each wood member at a joint and thereby eliminate the need of a lices in addition to joint connections. Also sach penel with its wood a abers and usset plates becomes a unit to which can be added similar units in tanks to provide adaptability to different s an lengths. In conjunction with the use of steel guassts, it is necessary to employ shear plates an the usans of transferring stress from wood to steel. This device itself offers . further advantage in that when it is installed it is flush with the face of the wood member and does not rotrude like the eplit ring. Thus a joint may be partially bolted up and the remaining mombers out be slic ed into place between the usest lates at a later time in the erection without any difficulty. A disadvantage in using steel at the joints is that it does not have the ability to successfully withstand high stresses of short duration as does wood. Jonsequently though an increase of 100 per sent in allowable stress is cersitted for impact loads in wood, the allowable stress in steel is unclaused whether deeling with impact loads or those of long-term duration. It remains to be seen whether this estuation will cause any major trouble in obtaining a suitable desi n.

In order to meet the requirements of varying stress aspecity in the different truss members and at the same time realize standardization to the maximum extent, a single wood section several of which could be febricated side to side to afford different load separations would offer the ideal solution. With this consideration is sind, preliminary member designs were attempted employing several basic

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timber sections. At the outset it was lisauvered that the connections required a commaratively large number of some lites. In the interest of caintaining a resonably small susset slate area, two rows of connectors instead of a sin-le row at the ends of the me bers was indicated. T is automaticall limited the basic me ber to a minimum width of 12 inches (nominal) to accommodate 4-inch at ar places. Winco me bers of renter width are more difficult to obtain in quantity, verious tolden esses of 12-ine clears were first investigated. Freliminary analysis resulted in the following conclusions. The 2" by 12" is structurally too s all. The 5" by 12", because of its hi h L/i retio for the len the involved, results in the copression me bere bein desi ned as long columns with sonsequent mes er allowable atr sees. The 4" by 12" produces an int rmediate column condition for the ter cord and verticals and at the seme time has a load capacity small enough to make multiples of the basic ber ala-table to a wide range of trues stres re-wire a to wit out unreasonable ov rdeal n in any particul r situation. Turth rsore, because of the fact that the 4° by 12° is atmost really feed the on also is the same section from which the deck is constructed, it is an are tionally (vorsb) choice fro the lo-istical musideration.

The preliminary computations lso resulted in two additional conclusions fich are incorporated in the subsequent that design.

The 4-inch shar plates as patented by the finite in incorporate, as a patented by the finite in incorporate as a special bubs to take either three-fruiths or siven-eithts incorporate bolts. Owners the increase of lifty for sent in the court value of the scar late for less all of a scalar five finites during causes a armin even the

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s ven-si bths inch bolt to be critical. In order to fully realize the hi her load can acity of the sor late it is necessary to incrosso the tolt sire to one inch. This in turn provides ore bearing area between bolts and ruseet plates and con amently perwits the use of thi mer plates t'an would otherwise be re-ulred. Also it was found during the reliminary investigation that a redu tion of the parellel to rain a main of shear lates to the ini um 5 inches is advante sous in reducing to area of the use t plates. The contraction of the section to the 5-insh inimum is per itted at the expense of reducing the load value of the story leles to seventy-five per cent of their full value. Since the s ar lates cour in two rows and on both faces and are therefore used in even multiples of four, the contracted a cin in some cases does not require any increase in the number of stear plates. Furthermore the 5-inch pesin adds anoth " feature of uniformity in the overall do in and thus si rlifies the borin of bolt holes.

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THE R. P. LEWIS CO., LANSING, MICH. LANSING, MICH.

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Allo mble Unit Stresses: -

| Tersion cralled to rain (t) Dempression peralled to crain (c) Dedulus of elasticity () | 1500 psi 1150 psi 1600000 psi |
|---|-------------------------------------|
| Steel - | |
| Shear for unfinished bolts | 13500 psi |
| Bearing for unfinished bolte | 28125 psi |
| *xial tension on net section | 27000 pai |
| Compression in gusset plates | 24000 psi |

Assume all members $4^n \times 12^n$ (A = 41.69 ag in). Use 1/2s pusset lates throughout. Use 4" shear lates with 1" bolt at 5" spacin throughout.

Load chart value of one A* shown plate (wood-to-steel) for angle of load to grain O degrees 6.56 k

Increased capacity of one shear plate ten desiming for dead load plus live load

$$6.56 \times 1.50 = 9.84 \text{ k}$$

Reduced caracity of one s'ear plate at 5" spacing parallel to rain

Value of one 1" bolt in single s. ear at the two faces of adjacent guesot plates

Value of one 1" bolt in bearing on half the width of two eljacent usset plates

$$2 \times \frac{1}{2} \times \frac{1}{2} \times 1 \times 25.125 = 14.06 \text{ k}$$

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$$K = 0.702 / \frac{1500000}{6} = 0.702 / \frac{1500000}{1150 \times 1.50} = 30.45$$

$$K_0 = 1.5811 \times 30.45 = 48.2$$

$$R_0 = 1.5811 R = 1.5811 \times 30.45 = 48.2$$

$$K_b = 1.7320 \text{ } = 1.7320 \text{ } \times 30.45 = 52.8$$

$$\frac{L}{d} = \frac{13 \times 12}{5.625} = 43.0$$

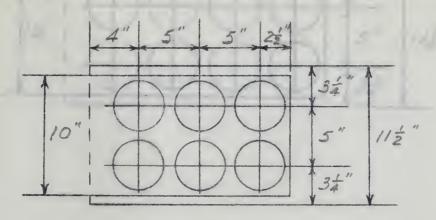
Assume space t column with oni condition "b"; therefore design as intermediate column.

$$o' = o \left[1 - \frac{1}{3} \left(\frac{L}{K_b d}\right)^4\right] = 1150 \times 1.50 \left[1 - \frac{1}{3} \left(\frac{43.0}{52.8}\right)^4\right] = 1.472 \text{ kg}$$

Try two ress of 5 shear plates each in both faces making a total of 12 shear plates and 5 belta.

- A. Jaracity due to compression parallel to grain in wood: 41.69 x 1.472 = 61.4 k ()L + LL)
- Capacity has to load value of shear plates: 12 x 7.38 = 88.5 k (DL + LL) D. (D. o. all or Time)
- U. Peracity du to bolts in shoer: 6 x 21.20 = 127.2 k (0b + Lb + 17)
- D. Capacity due to bolts in bearing: 6 x 14.06 = 84.4 k (DL + LL + TP)
- C. Sapacity due to co pression in 10° effective gueset width: $(10 - 2 \times \frac{17}{36}) \times \frac{1}{2} \times 24.0 = 94.5 \times (0.1 + LL + IM')$

Since limiting capacity of 84.4 k (L + LL + I') examds limiting capacity of 61.4 k (L 4 LL) by more than 30 , the latter loverns. A minimum of two basic member lest be u d to setisfy the requirement for a spaced column.



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Try 2 rows of 4 s ear plates each in both Roses waling a total of 16 shear plates and 8 bolts.

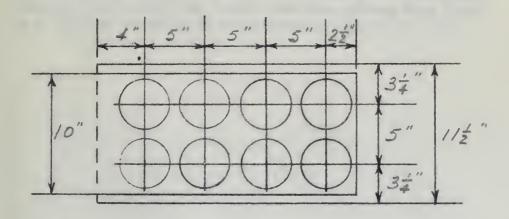
A. Japasity due to allowable stress in wood at inter disteres.

- 2. Depasity due to allo mable stress in wood at not section: $(41.69 2 \times 7.34) \times \frac{1}{.52} = 84.4 \times (0.14 \text{ LL})$
- J. Sanacity due to load value of shear plates:

. Dancity due to bolts in shear:

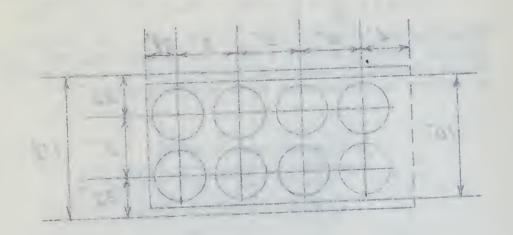
- 8 x 14.06 = 112.5 k (DL + LL + INI)
- F. Capacity due to tension in 10° effective caset width: $(10 2 \times \frac{17}{16}) \times \frac{1}{2} \times 27.0 = 106.3 \text{ k (L + LL + LL)}$

limiting capacity of 84.4 k (NL + LL) by only slibtly less tan 30, consider the latter governs.



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lince in street in it will is to mich, the errs besic ber as that in bottom chord is used with limiting capacity of I'. k (. + If). To ver e pres ive counter stres es may be diveloped and therefore its limiting capacity in compresion ust be letermined.

$$K = 0.702 / \frac{3}{6} = 0.702 / \frac{1600000}{1150} = 57.3$$

$$K_{a} = 1.5811 K = 1.5811 \times 37.3 = 59.0$$

$$K_{b} = 1.7520 V = 1.7520 \times 37.3 = 64.6$$

$$\frac{1}{4} = \frac{13 \times 12 \times 1.11}{3.625} = 60.5$$

$$\frac{L}{d} = \frac{13 \times 12 \times 1.41^4}{3.625} = 60.5$$

Assume spaced column with end condition "b"; therefore design s int re dista column.

$$3' = 0 \left[1 - \frac{1}{3} \left(\frac{L}{V_{\rm bd}}\right)^4\right] = 1150 \left[1 - \frac{1}{3} \left(\frac{60.5}{64.6}\right)^4\right] = 0.855 \text{ keV}$$

Capacity due to co pression rarallel to rain in wood:

Limiting value of 35.6 k (Di. + LL) abviously reverns over listin values due to abear plates, bol s, etc. A infaum of two basic cerbers must be used to stisfy requirement for a paced column. personal all residences and improve contains real-real value or annual

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MEMBER DESIGN

Verticals -

Since main stress in verticals is compression, the same basic member as that in the top chord is used with limiting capacity of 61.4 k (DL + LL). However tensile counter stresses may be developed and therefore its limiting capacity in tension must be determined.

- A. Capacity due to allowable stress in wood at intermediate section:

 41.69 x 1.6 x 1.50 = 100.0 k (DL + LL)
- B. Capacity due to allowable stress in wood at net section: $(41.69 2 \times 7.34) \times \frac{1}{.32} = 84.4 \times (DL + LL)$
- Capacity due to load value of shear plates: 12 x 7.38 = 88.6 k (DL + LL)
- D. Capacity due to bolts in shear: $6 \times 21.20 = 127.2 \times (DL + LL + IMP)$
- E. Capacity due to bolte in bearing: $6 \times 14.06 = 84.4 \times (DL + LL + IF)$
- F. Capacity due to tension in 10^8 effective guaset width: $(10-2 \times \frac{17}{16}) \times \frac{1}{2} \times 27.0 = 106.3 \text{ k (DL + LL + IMP)}$

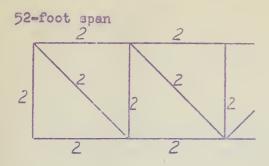
Since limiting capacity of 84.4 k (DL + LL + IMP) obviously governs, eliminating the impact portions reduces this figure to a limiting capacity of approximately 65.0 k (DL + LL).

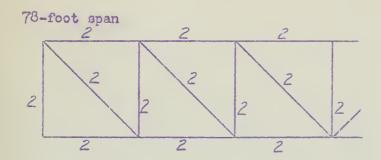
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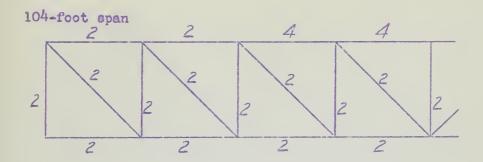
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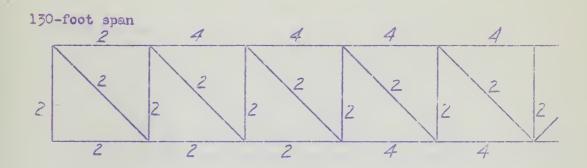
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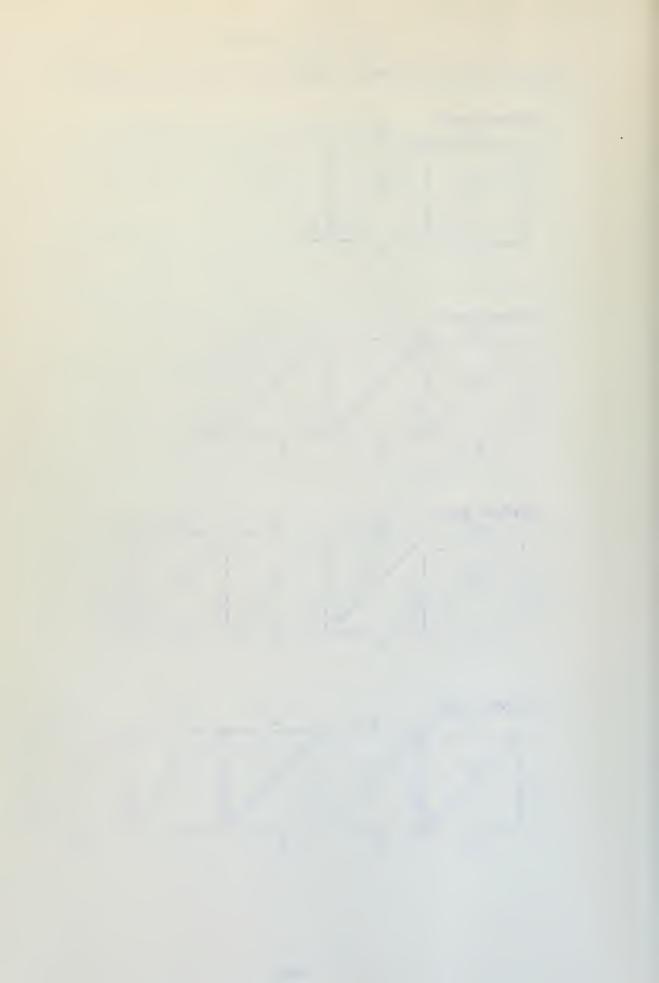
Number of basic components required for various truss members.



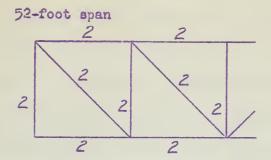


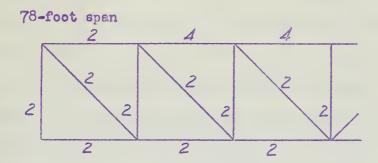


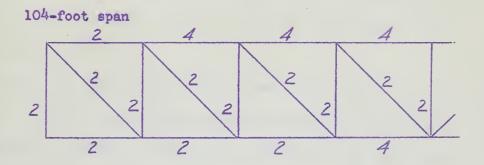


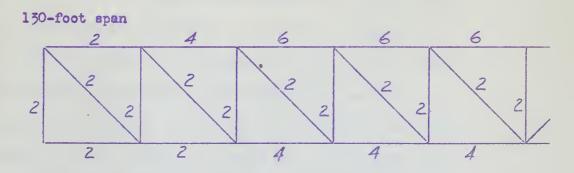


Number of basic components required for various truss members.











o. runs attils - To real of the trues do in i virtually soul to dat this point. It remins to leter ine the detrils of abritation. Ir t l t us co si'er the lower c'ord russet linte (71 . 3). It has been determined that all use to "ill be one olf inch thick. Therefore only its there and unatin schedule is now remired. All lo or clord joints are si ilar xospt for the center joint . . re there are two dis onels instead of one. It is improsticed to attempt to hear the fleer-beam onto the verticals; so t my mu t beer directly on to of the lover cord ussets. There will be a floor been on either side of the vertical. Consequently the usest must have a forirontal top says that extends at love ourte a to sixteen inches outside the edge of the verticals. As a result it is resented to the site of fill uration on the center foint with two in onals and for stand - igation to use the same plate at all lower stort joints.

In the case of the 11 ht briles to reserve the 12" b 15 looked been a him lie on ait) reside of the verticels. Or the havy been a third 12" looked to say between the two outer be a tith its ends receive the verticels. To effort a satisfactory bearing a reach, a U-shound high inch plate (i. 4) that slips around the verticel and boars on the top edges of the lower chard case to it willed. It is held in resition by a all an least on the inide of the verticels to seat the iddle floor-been in

II where many one has been part of a part of the The same of the sa the court and working on July many proposition his proposition the same processed post to the art will shall be seen the same and the second with the same of the the same named points (II). The threat was not a policy of the con-THE RESERVE THE PERSON NAMED AND POST OFFICE ADDRESS. will have not because of healthcome with any year to be be a report to be or planets are presently at principles of the contractly AND REAL PROPERTY AND ADDRESS OF THE PARTY AND PERSONS NAMED IN CO. OF THE PARTY AND PARTY AND PERSONS NAMED IN CO., NAMED IN CO NAME AND POST OFFICE ADDRESS OF TAXABLE PARTY AND PARTY. married and reserved to the party of the par to the second series of the larger was the paper and the bester becomes and the last of th I will have been sent our of purpose there has been bloomed. AND REAL PROPERTY.

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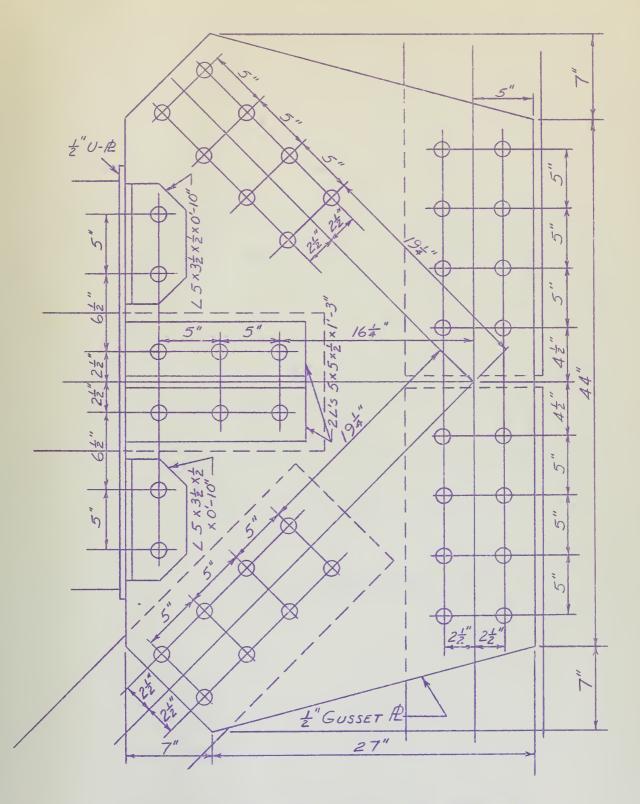


Fig. 3 Typical Lower Chord Joint with Stiffeners



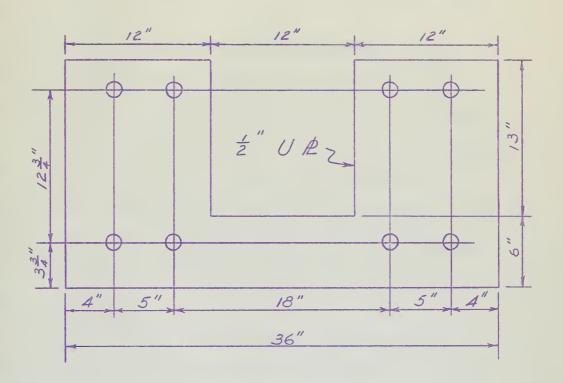


Fig. 4 Floor-beam Bearing Plate



the heavy bride. In the clusters were the better chord is the some total this make and the vertical it is recessary to bolt the aless to the inside race of the vertical to revise a stiffered and the remark. Here the selfer clarify than the vertical the interior waset supports the sent for the center loor-boss.

contion of the center joint. Again a single gueset (Fig. 5)
will be used in the interests of uniformity decite the fact that
there will be used essay protrucions at the center and and joints.
After existion the aperflucias portions could be burned off if
it were to irad to improve the appearance of the atrusture. Though
the stand of the quest is highly irregular and will be expensive
to out out, it is beened advantageous to make it so and thereby
reduces its a int.

subject to compressive stranges require spacer blocks of some and at the course let mean each of the basis members. The blocks are held in land to bolts on cause the component moders of the spaced column to est in union under stress. The spacer blocks may be either of cod planed down to half inch thickness or a half-inch drilled steel strep. The top chord and vertical members are primarily compression members; so they require a soon lines some lia onals are subject to counter compressive a reases, they too the little limit for a scen block helt. The botto chords of the little for a scen block helt. The botto

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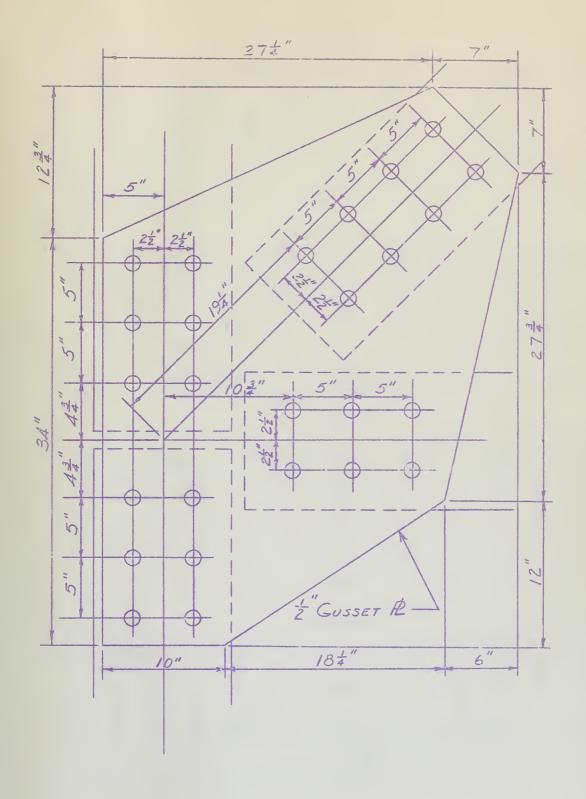


Fig. 5 Typical Top Chord Joint



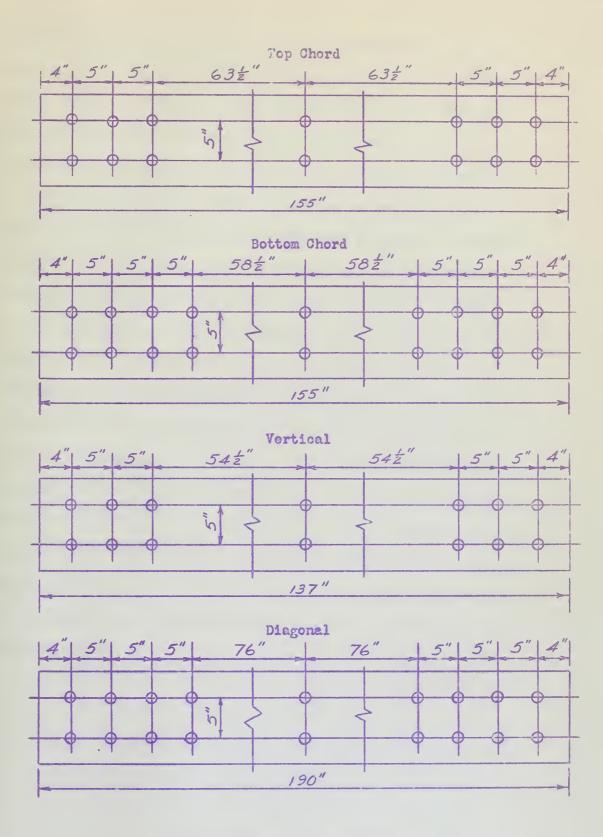


Fig. 6 Basic 4" by 12" Components



because their secondary stresses mi ht be compression, they must be provided with spacer blocks. The conclusion is that in fabrication all beside above will be drilled for two bolt holes at their next re; then after or ation all trues numbers expecting the interior bottom shord will be fitted with laser blocks.

that each to chord manel point is lawerally supported to rount buckling. Lateral support can be provided by extending one of the floor-beams through and beyond the trues about sever feet and then installing a 5° by 8° bracing struct from its end to the top shord. This arrangement can be ease plished conveniently because of the double floor-beams. The 12° by 12° on one side of the vertical is extended through the right trues and the other 12° by 18° through the left trues. Thus the maximum length of floor-beam needed is the contex-to-senter ensure of the truess plus seven feet whereas a single piece floor-beam would have to be extended at both ends and besides being hure in section would be extremely long. For the heavy bridge the longest floor-beam in the proposed design is a 12° by 18° two ty-six fort limit is in not unreason ble.

a silvery transfer serious as blaze from plants and in contrast.

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near part productions of phylo harded resiseas which received action of the bedders of a point of the last times of a point of the bedders of the point of that and the bedders of the point of that and the matter of the point o

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VI. SUPPLATION AND OF CHUSIONS

The problem involved in this thesis is an evaluation of the military timber bridge requirements for the J. S. Frine Corps and the evolve at of a sliberate plan by which these requirements can be most effectively met. The evaluation is intended to enswer the question of west different timber bridges are needed with regard to variation in load limits, type of construction, number of lands and span lengths. Thence the plan for satisfying the requirements enconverses the design of all the essential structures with special attention to facilitating their production, supply and erection by means of at many implies and uniformity.

The objective as ori inclly stated is "to predesign as far as practicable the semi-personant timber bridges which are most commonly employed by the ". . . origin force in military operations according to the very mand of treflic or ecity, lord or ecity and eits condition; and to determine the extent to mind standardization of construction details, structural design and common at materials required is feasible."

A study of the bridge requirements indicated that the type of construction most frequently needed is the timber treath bridge.

On infr quent seasions where the cite precludes the installation of a timber treath bridge, a timber trues bridge would be profitable in avoiding the use of a more specialized prefabricated metal bridge much as the Briley. It was further concluded that two different load capacities huld merico to provide passage for all combet vehicles.

The lighter capacity, nominally 35 tens, is besed on the style tank

The series arrows as a ready and advantages and the series and the

 tons permits passage of the MOS tank as the most severe load.

Torardiar the number of lanes it was decided that a sin le-lane readway is the more usual require ent but the lemand for a double-lane readway is frequent enough to warrant its inclusion in the decimal.

The attack of the deal m problem was preceded by the formulation of the design criteria which would overn. The "4 and "26 tenks were adopted without appreciable change as the design vehicles for the light and heavy bridges, respectively. Sased on t'e actual overall widths of the design tanks and erbitrarily chosen alerrances, the required cloor wilths of readway were det rai ed to be twelve and one half feet for the sin-le-lane li ht brid o, twonty-two foot for the double-lene li ht bride, fifteen and one helf fest for the sin le-lane heavy brid a and to mtyal ht feet for the double-lane heavy brid . The allowable unit stresses in wood were selected with the sim of safely utilizing the enjority of stress-rades of Touthern Time and Douglas ir lumber. An analysis of the loads of various duration in conjunction with the atte dent ingresses sermitted in allowable stress proved that it is safe to base dealer on two-thirds or wimm dead lus live load uning the basic allows le stresses and is act on thereby be impored. The ellowable unit stres as in steel were selected from pertinent Department of the Army publications and for military application are acceptat more liberal than those corres online in civilian practice.

the state of the best of the same of the s the same of the sa the first one or the female and the first owner where the same party makes of the or greatly may be a set or and all The same of the Assessment and the same of the same in contrast would be produced by the same of the state of the s and the second s the same of the same of the same of the representation and application of the part of the THE RESIDENCE OF THE PARTY OF T the reason has not been been been been as a proper of the party of the AND RESIDENCE OF PERSONS AND ADDRESS OF PERSONS AND ADDRESS OF THE PERSON ADDRESS OF THE PERSO word to the same of the latter of the same Section in charges in bright self all year of the regulations named the name of the party of the last to the party of t the state of the same of the s where the party was not been a finished by the party of t the contract of the party of th Capitality of filling at a filling of the filling o

In general the design computations followed the long tional procedures of accented to ber enjugaring. The only unitue feature of the design process was the judicious selection of mamber sections to promote standardization werever possible. The initial step was the determination of the maximum aran for decks consisting of 5° by 12" planks, 4" by 12" planks and 2" by 4" strips laminated. Thence reneral expressions for the required section modulus and area of stringers on a fifteen-foot span for structures having plank or in insted decks and one or two traffic lanes were formulated. y the use of these ex ressions sourled with considerable trial and error, it was found t at t s most advanta sous co bination of dec., atringer section and stringer a acing is a 4" by 12" plank dock and 8" by 16" stringers at a seings of ?" and 22" for the 11 ht and heavy bri co, respectively. This means that all bri es ithin the score of this investigation, regardless of loni co esity, number of I nes or ty o of con truction, will ov to exnot seed dook and the same size stringers. The only difference in the light and heavy bridges of either width and any mode of construction is the s, acinof the strin ers.

The design of the treatle basts would we been to logical attractural commonent to investibate a xt. To verit was felt that the words variables off ating their design aveclittle regise of rotified attains in tion. To see up thy the current ractice of roviding heavy timbers of 10° by 10° size and larger to serve as sills, seets and can an accested without any attent at improve ont. Thus the efforts toward standardization as in hit pertain

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 directly to the troutle bridges were completed.

the true structure was undertaken. Because of the unresponsible its stiens that all be required or floor-bears in bubbelene true widges of the light a well as the lavy local achority, the dain of trues bridges was limited to those of sin le-lens width only. The investigation of floor-bears resulted in the slotton of a 12 by 13° as the most as regulated in the surpose at and. Utilizing two of these bears per and reint the floor-bear requirement for the light trues bridge and three boars of the exact series estion serve the purpose in the heavy trues bridge. Thus only one size timber is required to perform the function of the loor-bear in either the light or heavy trues bridge.

the lesion of the various length trustees for both to light and have bridge was then undertaken. The type of truss besided upon as other than the prediction is a subject to the length. It is the context of the light end on a light decreased triangle on a true foot screen on results in a light decrease of every interior very it so as resemble to exait design scene, you iver you to the interior of context is a light decrease of every to the interior of context is a light decrease.

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the lan of single basic section throughout ill trusses.

The len features the use of basic comment loces its by aid in

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russets on the ortaide and between each of the emmonent pieces transmit the strasses at a joint in unjunction with the use of shear lates as connectors. The basis oction linally a lected is a 4° by 12° die entails no addition to it as already appear—in on the amount of all truss and the very same section found in the each. Ince all truss and the can be derived from sixteen feet wide, 4" by 12° by 16° pieces may be provided and used indiscriminately as either dect clarks or components of truss me bers in either sei ht class bridge.

The lifting values of the 1st by 12st basic piece used as a component of to shord members, bottom shord members, vertical members and disconal members were each det rained. Trance the deal n of any truse, light or heavy, short or long, consists merely of dividing the sector trans by the applicable limiting value of the 4st by 12st basic pice to determine the size of the 4st by 12st basic pice to determine the size of the deviation of the deviat

Three different steel lates, all of helf inch thickness, are required in addition to the wood water to combet the trusses.

One plate serves as the lower chord usest, another is the urper chord susset and the third is a bearing late for the floor-beau.

These plates serve their purpose in either the light or heavy truss.

Another feature of standardization incorporated in the true design

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